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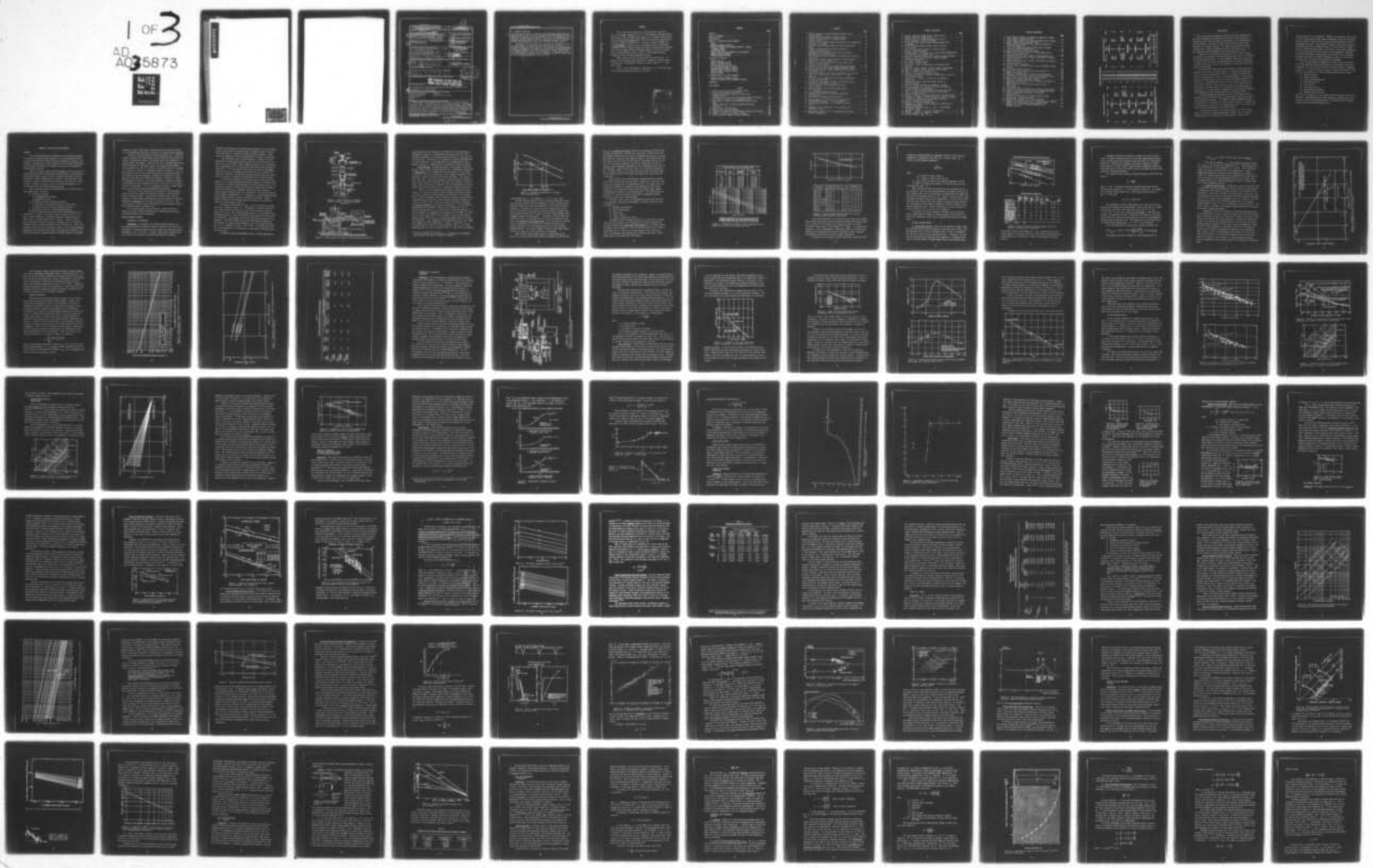

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This report volume presents a state of the art for fatigue cracking of pavement systems. The report deals with prior research relating to the fatigue of asphalt concrete, asphalt emulsions, cement-modified emulsions, lime-treated material, lime fly ash, lime cement fly ash mixes, cement-treated material, and portland cement concrete. A comprehensive summary touching upon laboratory test procedures, test methods, influence of mix factors upon fatigue, generation of typical fatigue curves, development of fatigue fracture subsystems, and verification studies is presented. (Continued)

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20. ABSTRACT (Continued).

The state of the art is summarized and qualitatively rated relative to eleven different features felt to be necessary for the development of a fatigue subsystem. In addition, this summary formed the basis for recommendations dealing with future research on fatigue. The basis of the fatigue subsystem is formulated upon the concept that any such methodology proposed for fracture distress must ultimately be founded upon a functional pavement failure.

Accordingly, the major results for future research, regardless of material type, appear to be focused upon the development of a distress-to-performance model that can be accurately used in pavement design and management systems. In addition, the development of a probabilistic fatigue methodology is likewise considered to be a major research area. Finally, while several laboratory-type research studies are recommended, it is suggested that most of the research effort be devoted to field performance verification studies using the wealth of existing knowledge already available.

PREFACE

This is the first volume of a two-volume report written by Dr. M. W. Witczak, employed by the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, as a pavement expert during the period May 1974-November 1975. This work was jointly sponsored by the Office, Chief of Engineers, U. S. Army, as a part of RDT&E Project 4A762719AT40, "Mobility, Soils, and Weapons Effects Technology," Technical Area A2, Pavement Systems and Lines of Communications Engineering," Work Unit 006, "Develop Performance Models," and by the Federal Aviation Administration (FAA), as part of Inter-Agency Agreement FA73WAI-377, "New Pavement Design Methodology."

WES personnel directly concerned with this work were Dr. W. R. Barker, project engineer and research civil engineer, Pavement Design Division, Soils and Pavements Laboratory (S&PL), and Mr. J. P. Sale, Chief, S&PL.

COL G. H. Hilt was Director of WES and Mr. F. R. Brown was Technical Director during the conduct of the work.

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FIGURES (CONTINUED)

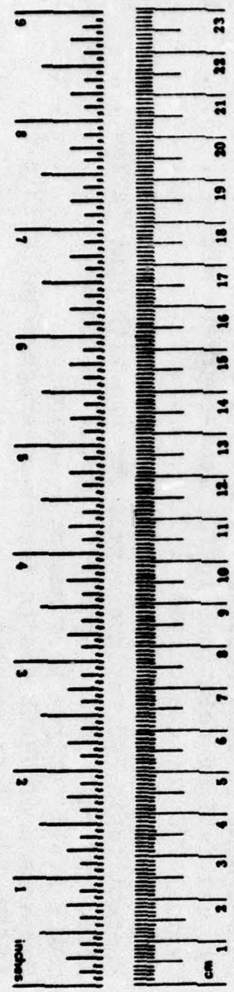
FIGURES (CONTINUED)

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

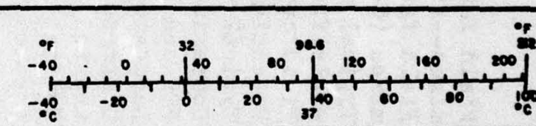
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



INTRODUCTION

This report volume describes the state of the art regarding distress due to repeated load applications causing fatigue cracking in pavement systems. Such distress is attributable to the development of tensile stresses or strains at levels that normally are below those necessary for ultimate fracture. The repeated applications of these stresses, due to loads, progressively lead to the fatigue life of a given material being exceeded and hence cause fracture to occur.

In general, the likelihood of this distress is directly related to pavement characteristics and load conditions that result in increased tensile stresses (strains). As such, pavements having very high moduli (stiffness) near the surface of the pavement show a higher probability of fracture and, hence, loss of "distributing or slab power" than lower moduli type materials.

At present there are a great many different design methodologies available that consider fatigue fracture. In general these approaches range from laboratory-developed criteria to criterion developed principally from performance studies of existing pavements. The structural analysis of fatigue has, historically, been more widely recognized and considered in rigid pavement analysis than in flexible (asphalt) pavement design. However, in the last 10 to 15 yr, much research has focused upon the fatigue behavior and development of structural design subsystems for not only flexible pavements but other stabilized material types subjected to the possible conditions of fatigue fracture.

In the assessment of the state of the art of fatigue fracture, one must view this distress (and all other modes of failure) with different design philosophies. For example, one must understand whether the discussion is a question of a "safe design" procedure or whether one is talking about the ability of a design methodology to "accurately predict the future performance" of a pavement system.

Although both concepts are design oriented, the former is much easier to obtain and therefore possesses a higher "state-of-the-art" rating because of its simplicity of achievement by safety factor

concepts applied to all parameters. However, this approach, while sound for other engineering designs, leads to excessive costs and, furthermore, provides little, if any, ability to predict deterioration and, hence, performance with time. In the author's opinion, this latter concept is absolutely mandatory if pavement design is to ever achieve a "higher step" in rational design concepts. As a result, the overall interaction of initial fracture prediction, rate of crack propagation, subsequent distress-to-performance relationships, and a failure level based upon functional concepts is considered necessary in order to truly define a procedure that can predict future pavement performance.

This volume attempts to summarize the state of the art relative to how well pavement technology addresses the "performance prediction" philosophy rather than just the ability to predict fracture. Beginning on page 11, an extensive summary of fatigue research and methods is presented. This summary is for the following pavement material types:

a. Asphalt concrete.
b. Asphalt emulsions.
c. Cement-modified emulsions.
d. Lime-treated soils.
e. Lime fly ash.
f. Lime cement fly ash.
g. Cement-treated materials.
h. Portland cement concrete.

Major results brought out by the current review of fatigue fracture are discussed. Recommendations for future research are given based on the ability of current design methodologies to accurately predict fatigue cracking and subsequent performance loss.

SUMMARY OF PREVIOUS FATIGUE RESEARCH

GENERAL

The following information represents a state-of-the-art summary of previously reported fatigue research on various pavement materials. As will be observed, much research has been conducted on repeated load cracking. Unfortunately, results frequently differ between researchers; this has been interpreted by the author as one source of potentially needed research activity.

There is, however, almost universal agreement that the results of fatigue resistance may be formulated by some functional relationship between tensile stress, tensile strain, or stress-to-strength ratio and the number of cycles (applications) to failure. This functional relationship is normally treated as being linear on a log-log plot or semi-log plot. Normally, the type of plot or relationship appears to be related directly to the stabilized material type.

These functional fatigue relationships may be obtained in one of three general ways. They are:

a. Laboratory testing.

  (1) Phenomenological studies.

  (2) Mechanistic studies.

b. Performance studies of existing pavements.

  (1) Empirical relationships (design or safety factor).

  (2) Theoretical relationships (stresses, strains).

c. Theoretical analysis of existing design methods.

As noted, laboratory tests may be either "phenomenological" or "mechanistic" in nature. The former type of laboratory tests, in essence, simply formulates a relationship between an applied stress or strain level to observed repetitions to fracture. Inherent in this type of laboratory testing is the type of fatigue test to be conducted (constant stress or load and constant strain or deflection). Also, this approach provides no direct information relative to the crack-propagation properties of the material under various environmental conditions. The mechanistic approach, although youthful in fatigue research for pavement

materials, offers a great deal of future potential to directly account for the disadvantages previously noted for the phenomenological approach.

Because of the uncertainties arising from direct use of laboratory results to predict field performance, other researchers have used the approach of determining a fatigue relationship based upon examination of actual pavement performance. While such a method circumvents many of the problems inherent in laboratory studies, the primary disadvantages of such an approach are that only "effects" are observed and little information relating to "causes" or fundamental behavior is obtained. Also, in many cases, the results have been obtained with a particular pavement mix composition and the subsequent extrapolation of predicting performance for other materials (mixes) may be subject to question. However, this procedure does possess the unique advantage of developing fatigue relationships for various predefined levels of "failure" or, in essence, "levels of performance." As such it possesses a powerful research capability in developing distress-to-performance observations.

The use of theory combined with an analysis of existing design methods does afford a general fundamental approach to develop typical types of fatigue criteria. However, these criteria suffer from the necessity of using design input parameters (such as moduli and effective temperature) <u>identical</u> with that used to develop the criteria. Thus, the flexibility of using these relationships for other pavement compositions (material types) is normally lost.

The following summary contains fatigue research conducted on asphaltic mixtures, lime-treated materials, cement-treated materials, and portland cement concrete pavements.

ASPHALT CONCRETE MIXTURES

UNIVERSITY OF NOTTINGHAM

<u>Background.</u> As defined by this state-of-the-art report, the following discussion of bituminous fatigue research by the University of Nottingham is concerned primarily with the studies and findings reported principally by P. S. Pell and S. F. Brown. While other coauthors have been noted in the literature with these individuals, the salient

laboratory investigations concerning fatigue appear to have been headed by Pell, while Brown has published papers primarily aimed at fitting Pell's results into a general structural analysis framework. References 1-13 summarize the salient publications of these two researchers.

The laboratory study of fatigue behavior of bitumen and bituminous mixes by Pell was initiated in the late 50's and has been culminated by a recent paper[13] which summarizes nearly 15 yr of laboratory study encompassing over 50 different mix types and several thousand fatigue tests. It is the author's opinion that the work at Nottingham, along with the work of Monismith and associates at the University of California (Berkeley), has provided the most extensive findings concerning the influence of bituminous mix properties upon fatigue fracture.

The laboratory fatigue studies conducted at the University have primarily used a rotating bending cantilever machine for constant stress testing and a torsional strain machine for constant strain testing. Additionally, because of inherent difficulties in making direct stiffness (moduli) determinations on specimens undergoing fatigue testing, a special dynamic stiffness device was constructed. This machine measures mix stiffnesses independently (i.e., separately) of any measurements made during fatigue testing on similarly fabricated specimens. Figure 1 shows the details of the fatigue system and Figure 2 illustrates the dynamic stiffness measuring device.

The fatigue system shown in Figure 1 is for the rotating bending cantilever (constant stress) test. The specimen is mounted vertically on a rotating cantilever shaft and a point load is applied through a bearing at the top. This loading system results in a sinusoidally varying bending stress of constant amplitude at any particular cross section of the specimen. The maximum stress occurs just below the neck of the specimen. A controlled temperature bath, using either an aqueous alcohol solution or water, allows temperature to be controlled to within $\pm 0.2°C$ with a temperature range of $-5°C$ to $30°C$. A variable speed motor also allows specimens to be tested over a speed range of 80-3000 revolutions per minute.

For dynamic stiffness measurements (ratio of stress amplitude to

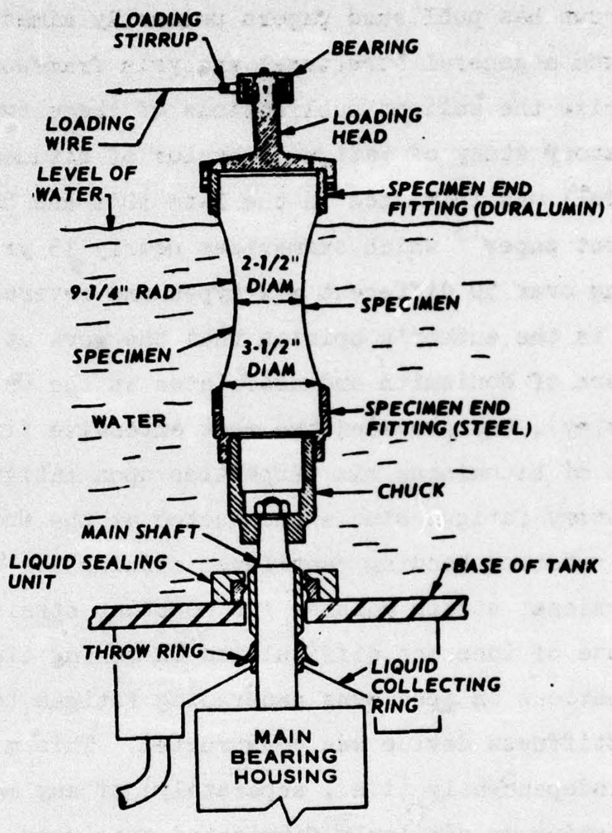


Figure 1. General details of rotating bending constant stress fatigue machine (from Pell[1])

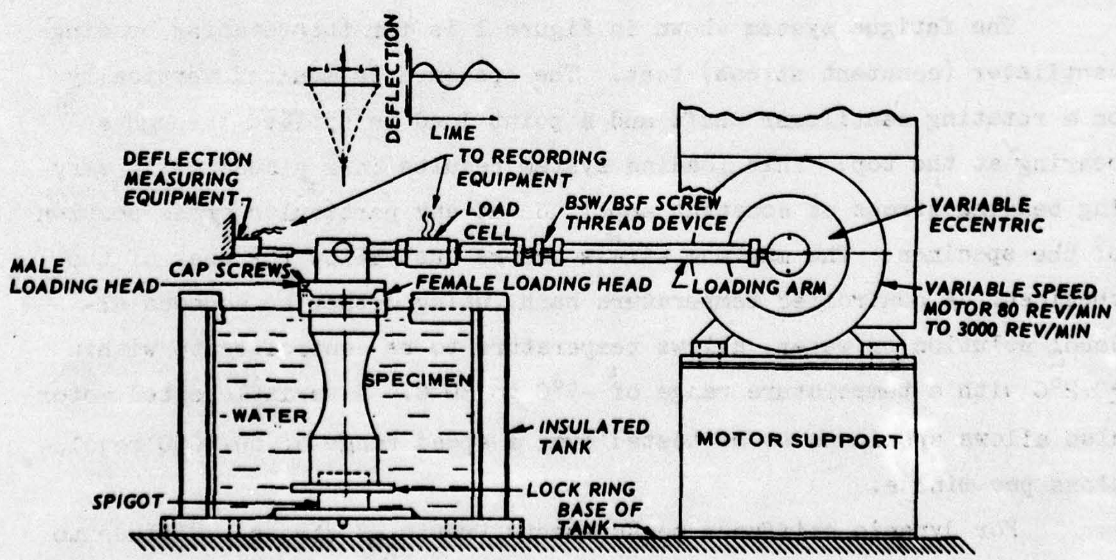


Figure 2. General details of dynamic stiffness machine (from Pell[1])

strain amplitude), the specimen is stationary and a sinusoidal type of deformation is applied. The deformation occurring at the top of the specimen is measured by a vibrating pickup device while a load cell monitors the force. Calibration of the machine is required using materials of known stiffnesses. It has been noted that the mean value of the dynamic stiffness of a particular mix is accurate to about ±10 percent.

Major Findings. Although much work in both constant stress and constant strain fatigue testing has been done at Nottingham, the majority of implementable work in fatigue has been concentrated in the constant stress mode. This has followed due to Pell's general acceptance of the Mode Factor approach advocated by Monismith and Deacon. This concept shows that, from a theoretical analysis of layered pavement systems, asphalt concrete pavements generally less than 2 in.* approach a constant strain condition (relative to laboratory test considerations) while bituminous layers greater than about 6 in. approach a constant stress mode.

In order to better understand the results of fatigue study presented, it is imperative that the reader comprehend the subtle distinction between the terms "fatigue damage" and "strain-life relationship." Figure 3 shows in schematic form two different fatigue curves plotted as a linear log-log relationship between tensile strain and repetitions to failure (or service life). The curves labeled AA' and BB', represent two distinctly different "strain-life relationships." The term fatigue damage refers to the number of repetitions to failure (N_i) due to a given strain magnitude (ε_i) for a particular "strain-life relation." If the unit damage (d) is defined as the reciprocal of the number of repetitions to failure $\left(d = N_f^{-1}\right)$, then from Figure 3 it can be seen that an imposed tensile strain of ε_1 will cause more damage with AA' strain-life curve $\left(\text{i.e., } d_{1A} = N_{1A}^{-1}\right)$ than strain magnitude ε_2 $\left(\text{i.e., } d_{2a} = N_{2A}^{-1}\right)$ on the same strain-life relationship. However, if the strain ε_1 is associated with the strain-life relationship BB', while ε_2 is still associated with AA', then the tensile strain ε_2 may result in a greater

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 8.

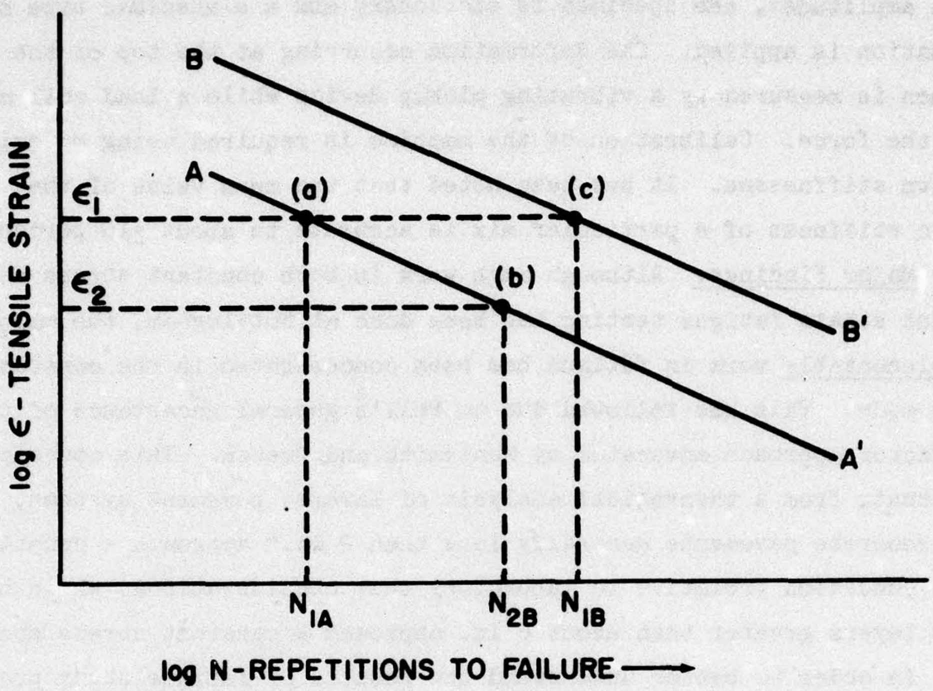


Figure 3. Schematic representation of constant stress fatigue results

fatigue damage than ε_1 even though $\varepsilon_2 < \varepsilon_1$ (i.e., $d_{2B} > d_{1B}$).

The most salient findings of Pell's intensive study for constant stress testing revolve around the importance of the mix stiffness as it influences fatigue life plus the development of his "strain criterion" hypothesis. In essence, the strain criterion is based upon the fact that he found several bituminous mix and test condition variables that only influence the magnitude of strain (and hence fatigue damage) along a <u>unique</u> "strain-life relationship." In contrast, other mix characteristics exist that also influence the location of the strain-life function or the <u>position of the fatigue curve.</u> Pell found that for constant stress testing many of the mix and test condition variables can be uniquely accounted for in the analysis by use of the mix stiffness (dynamic modulus). As a result a general review of the fatigue effects of both test conditions and mix properties is presented.

When constant stress fatigue tests for a particular mix are plotted on a log-log stress versus repetition diagram, the results show

that, for a constant stress level, mixes with decreasing stiffness yield increasing fatigue life. However, Pell found that a unique strain-life relationship occurred when the effects of temperature and rate of loading were taken into account by the dynamic stiffness. This demonstrated that the tensile mix strain was the major failure determinant and that the effects of the testing conditions, relative to temperature and speed of loading, could be primarily explained by their subsequent effect upon the mix stiffness. Figure 4 shows this result for a particular bituminous mix. As noted in the plot, the test temperature range varied from about -0.5°C (+32°F) to +25°C (77°F) while the loading rate varied from 1000 to 3000 rpm.[1]

Later work[8] on other mixes generally substantiated the validity of the "strain criterion" for most mixes. However, Pell noted that some evidence was present that for temperatures above 77°F, longer lives were obtained that couldn't be explained by the hypothesis of the strain criterion.[8] The concept was later verified for effects upon fatigue relationships from various confining pressures.[13] For these tests, use was made of a repetitive axially loaded tension/compression machine using 4-in.-diam by 8-in.-high specimens.

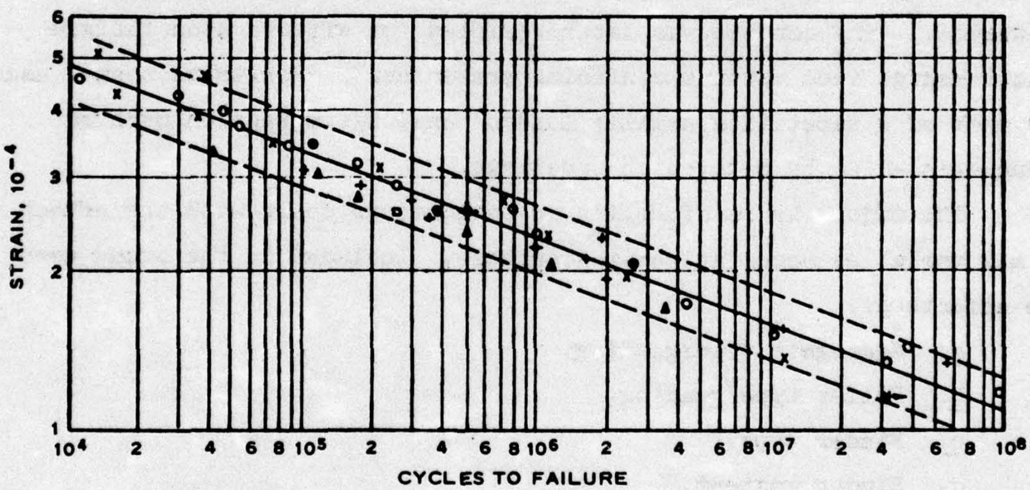
The major thrust of Pell's investigations dealt with the effect of mix variables upon fatigue performance. Included in the study were the effects of:

a. Aggregate type/grading.
b. Filler type/grading.
c. Binder type.
d. Binder content.
e. Degree of compaction (voids).

Although it must be recognized that all of these factors studied affect the mix stiffness, Pell concluded that the most important mix variables affecting the strain-life relationship were the binder content and degree of mix compaction. This is illustrated partially by Figure 5 which shows that for mixes with the same general binder content and voids, the aggregate type, grading, and bituminous grade have a negligible effect on the strain-life relationship.

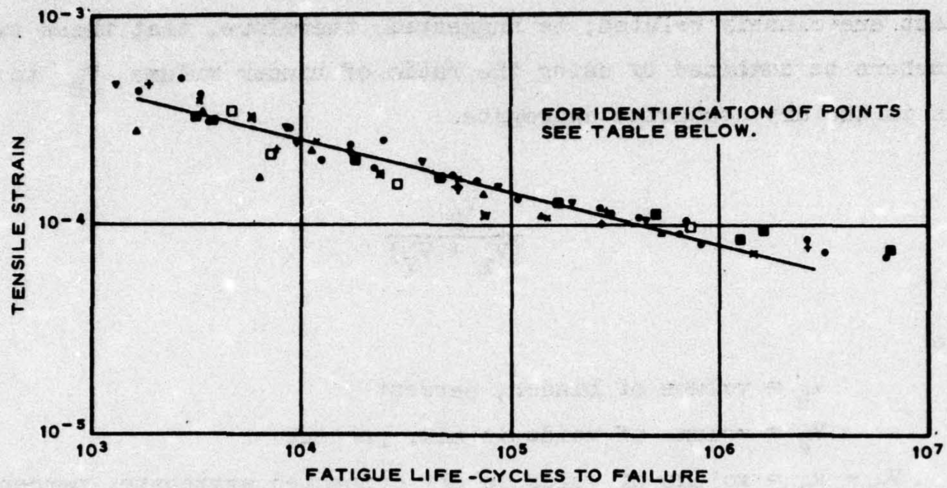
STIFFNESS OF SANDSHEET MIX, OBTAINED FROM PENETRATION INDEX AND RING AND BALL TEMPERATURE

TEMPERATURE °C	SPEED RPM	STIFFNESS OF BITUMEN NEWTONS/M²	STIFFNESS OF MIX 81% Cv LB/IN.²
-13.5	1000	9.60×10^8	2.90×10^6
- 9.5	3000	9.00×10^8	2.61×10^6
- 0.5	3000	4.10×10^8	1.81×10^6
- 0.5	2300	3.85×10^8	1.74×10^6
- 0.5	1000	3.00×10^8	1.57×10^6
+ 7.0	2300	2.10×10^8	1.28×10^6
+20.0	3000	9.00×10^7	6.80×10^5
+25.0	3000	5.50×10^7	4.65×10^5



TEMP, °C	+25	+20	+7	−0.5	−0.5	−0.5	−9.5	−13.5
SPEED, RPM	3000	3000	2300	1000	2300	3000	3000	1000
KEY	●	□	x	+	o	+	+	▲

Figure 4. Constant stress fatigue results showing effect of temperature and speed of loading (from Pell[1])



COARSE AGG. % by weight	FINE AGG. % by weight	FILLER % by weight	BINDER % by weight	MEAN VOIDS %	SYMBOL
60% c/rock	34% sand	0	6% 45 pen.	5.0	●
60% c/rock	34% c/rock	0	6% 45 pen.	5.6	○
60% c/rock	34% sand	0	6% 35 pen.	4.2	⌘
65% c/rock	29.3% sand	0	5.7% 45 pen.	4.0	✚
60% o/rock	34% sand	0	6% 100 pen.	4.8	⊙
60% quartz gravel	34% sand	0	6% 45 pen.	5.3	■
60% quartz gravel	34% c/rock	0	6% 45 pen.	6.0	□
65% quartz gravel	27.4% sand	2% cement	5.6% 45 pen.	3.5	+
60% flint gravel	34% sand	0	6% 45 pen.	4.8	▲
60% flint gravel	34% c/rock	0	6% 45 pen.	6.9	△
60% slag	33.7% sand	0	6.3% 45 pen.	5.0	▼
60% slag	33.7% c/rock	0	6.3% 45 pen.	5.6	▽
55% slag	38.2% sand	0	6.8% 45 pen.	5.4	×

Figure 5. Constant stress fatigue results for various bituminous mixes (from Pell and Brown[7])

A statistical analysis of all constant stress fatigue data showed that the most significant factors affecting the stress-life relationship (i.e., location of the fatigue curve) were: (a) binder viscosity, (b) binder content, and (c) void content.[8,13] Because the fatigue relationship (ε - N) is independent of temperature effects, an equiviscous measure of the bitumen was selected to characterize binder viscosity. The parameter selected was the ring and ball temperature.

In addition, Pell reported in 1973[8] that binder content and void

content are closely related; he suggested, therefore, that these two parameters be combined by using the ratio of binder volume V_B to voids in the dry compacted aggregate.

$$\frac{V_B}{(V_B + V_V)}$$

where

V_B = volume of binder, percent

V_V = volume of voids in mix, percent

$V_V + V_B$ = volume of voids in dry compacted aggregate, percent

In 1975[13] he changed this parameter to reflect only the volume percentage of binder, V_B.

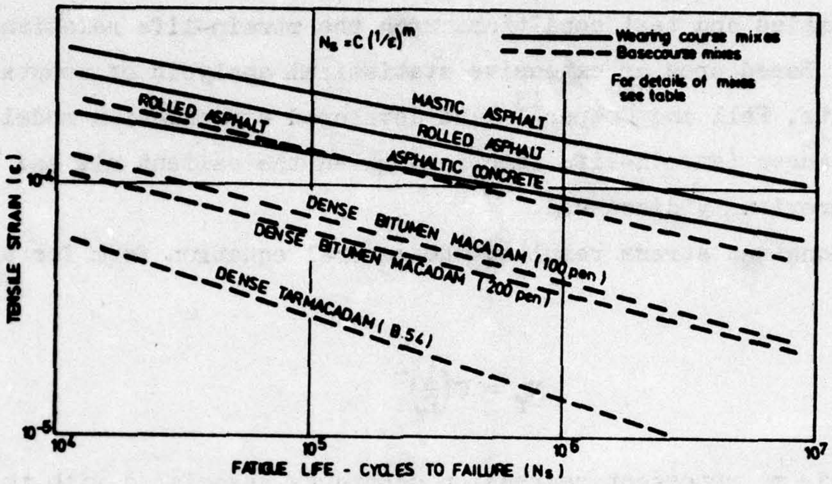
Thus from Pell's study the following can be concluded relative to mix variables. While all mix variables affect the dynamic stiffness and hence tensile level, the only significant variables that will change the strain-life relationship (see AA' or BB', Figure 3) are the binder volume and binder viscosity (ring and ball temperature). If these two mix variables are kept constant, all other changes in testing and mix variables will only result in an increase or decrease of the mix stiffness along a unique strain-life curve. This is equivalent to moving from point (a) to (b) or from (b) to (a) on curve AA' of Figure 3.

As a result, it can be concluded from Pell's work on constant stress testing that mix design for maximum fatigue life should be concerned with creating as stiff a mix as possible consistent with minimum voids.

TYPICAL FATIGUE CURVES

A. Lab-Constant Stress. Based upon the intensive fatigue study, Pell and Brown have presented laboratory controlled stress results that have applicability to typical bituminous mixes. Figure 6 illustrates one such set of typical curves developed by Pell. As shown on the figure, the appropriate mix properties are summarized. Of extreme importance is the wide range in fatigue damage between the bituminous mixes

Strain-life fatigue results for various mixes from controlled stress testing.



Typical mixes tested in controlled stress.

Description of Mix	Coarse Aggregate[a] (percent by wt)	Fine Aggregate (percent by wt)	Filler (percent by wt)	Binder: Percent by Wt	Binder: Penetration	Mean Voids (percent)	C	Slope Factor
Mastic asphalt wearing course	42	23[b]	30[b]	15	70/30 TLA/20	0	1.13×10^{-15}	5.5
Rolled asphalt wearing course, BS 594, gap graded	30	52.2[c]	8.9[b]	7.9	45	2.9	8.8×10^{-15}	5.1
Asphaltic concrete wearing course, continuously graded	42	46.8[c]	4.7[c]	6.5	70	3.6	2.2×10^{-10}	6.1
Rolled asphalt base course BS 594, gap graded	65	29.3[c]	–	5.7	45	4.0	6.7×10^{-12}	4.2
Dense bitumen macadam base course, MOT spec., continuously graded	62	28.6[c]	4.7[c]	4.7	100	6.8	1.9×10^{-11}	3.8
Dense bitumen macadam base course, MOT spec., continuously graded	62.3	28.7	4.7[c]	4.3	200	6.9	1.8×10^{-12}	4.0
Dense tar macadam base course, MOT spec., continuously graded	61.7	28.4	4.7[c]	5.2	B 54	7.5	2.7×10^{-9}	3.0

[a]Crushed rock. [b]Limestone. [c]Sand.

Figure 6. Constant stress laboratory fatigue results for typical bituminous mixes (from Pell[8])

at any given tensile strain magnitude. Such a characteristic should firmly implant in the reader's mind the limitations associated with assuming "typical" or "provisional" fatigue relationships for design analyses.

Although the results illustrated in Figure 6 by Pell are quite useful, his major contribution lies in the study summarizing the effects of mix properties and test conditions upon the strain-life relationship prediction. Based upon an extensive statistical analysis of constant stress results, Pell and Cooper[13] have developed a prediction model for the fatigue curve (strain-life function) given the salient mix and binder properties previously discussed.

For constant stress results, the general equation form for all mix types is:

$$N_f = C\left(\frac{1}{E}\right)^m$$

where C and m represent regression constants associated with the particular mix tested and E is the modulus of elasticity. The analysis of the fatigue results showed that a relationship existed between m and C of the form:

$$m = 0.5 - 0.313 \log C$$

The usefulness of such a relationship lies in the fact that all fatigue lines (strain-life relationships) for all mixes meet at a common focal point. For Pell's work, this focal point was $N = 40$ and $\varepsilon = 6.3 \times 10^{-4}$ in./in. (i.e. 630 microinches strain). Thus with this concept, the prediction of the N (repetitions) to achieve a particular strain level as a function of mix properties allows the establishment of the unique mix strain-life curve. The magnitude of strain used by Pell in the prediction equation was $\varepsilon = 1 \times 10^{-4}$ (100 microinches).

In a 1973 publication[8] the predictive equation developed was:

$$\log N_{\varepsilon=10^{-4}} = -16.34 + 6.03 \log\left(\frac{V_B \times 100}{V_B + V_V}\right) + 5.99 \log\left(T_{R\&B}\right)$$

The equation was later modified in a 1975 publication[13] to:

$$\log N_{\varepsilon=10^{-4}} = -11.13 + 4.13 \log V_B + 6.95 \log \left(T_{R\&B}\right)$$

In the latter equation, V_B is the volume percentage of binder and $T_{R\&B}$ is the ring and ball temperature in C°. The multiple correlation coefficient of this equation is $R = 0.936$. Thus it explains almost 88 percent of the variation in the prediction of $\log N$. A simplified nomographic solution has been developed and is shown in Figure 7.[13] Thus by knowing the two mix properties, the unique strain-life relationship can be ascertained by the use of the figure. This eliminates the need for extensive laboratory testing for fatigue analysis of most bituminous mixtures.

B. <u>Adjusted Field Curves.</u> It has been previously noted that even if the constant stress laboratory test exactly models the field behavior of bituminous mixes relative to crack initiation at the bottom of the bound layer, it will not correspond to in situ failure conditions. This is so because the lab tests do not reflect the three important characteristics of: (a) vertical crack propagation time, (b) effects of rest time between stress pulses, and (c) the time required for a structurally cracked asphaltic layer to reach a "functionally" failed condition. This latter item is, in essence, the required relationship between structural distress and pavement performance.

Although little if any information is quantitatively known about the increased time (or strain repetitions) that occurs from crack initiation to functional failure, it can be safely concluded that each of these effects will increase the actual repetitions to failure from laboratory results. To overcome this possible degree of conservatism, various adjustment factors (constant "N" multipliers) have been suggested. Brown and Pell[6,10] have suggested in 1972 that a factor of 20 be applied to account for the effect of intermittent loading and crack propagation time. Brown, in 1974,[11] has indicated that a factor of 5 may be warranted for the effect of "rest periods" between actual stress pulses along with a factor of 20 to account for crack propagation effects. Thus, the suggested overall combined factor of 100 has been advocated by Brown.

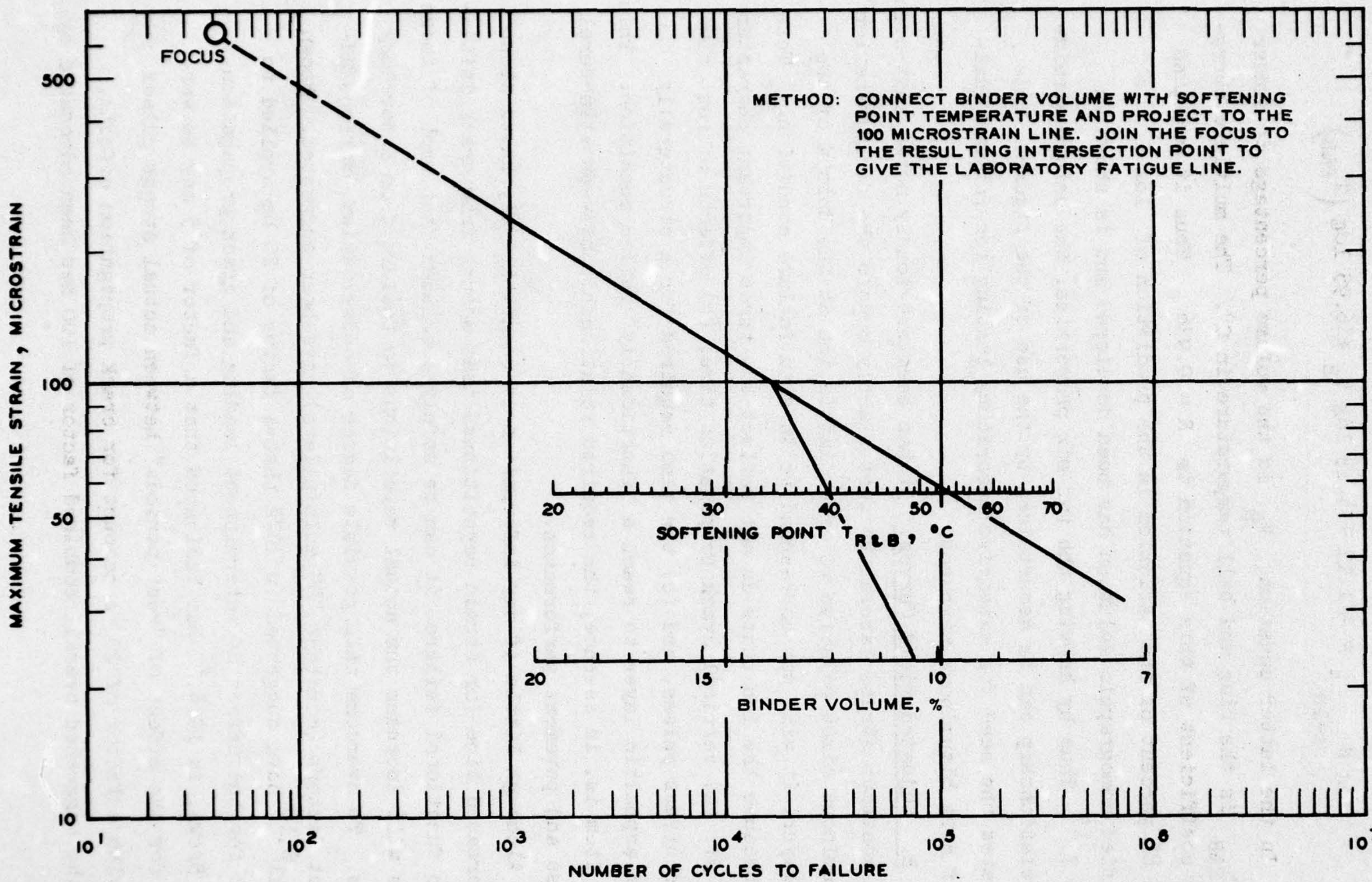


Figure 7. Nomograph for the prediction of laboratory fatigue performance of bituminous mixes (from Pell and Cooper[13])

Figure 8 shows a typical (provisional) asphalt concrete fatigue curve that incorporates an adjustment factor of 20 into the curve.[6] Thus this field curve, initially based upon lab fatigue tests, has been modified to field (actual design) use. Figure 9 illustrates a later (1974)[11] version showing field curves for three different asphaltic mixtures. These curves have been modified by a factor of 100. Table 1 summarizes the basic properties of each of the mixes noted. These curves have been developed from Pell's prediction equation previously discussed for lab results and adjusted by the 100 factor conversion.

DESIGN APPLICATIONS

Lab-controlled stress results with or without the field adjustment factor can be used to directly analyze fatigue. Because of the strain-criterion hypothesis, there is no "effective" or "critical" temperature (stiffness) at which the strain must be evaluated. Hence the unique fatigue is considered applicable for all environmental conditions and, if desired, cumulative damage theory can be directly employed. In publications dealing with the use of Pell and Brown's fatigue design curves, it has been noted that a pavement temperature condition corresponding to the general mean annual air temperature has been used. Although this may be applicable to environmental conditions existent in England, blanket use of such a design analysis is not to be recommended. Brown[11] has also suggested that the traffic weighted mean stiffness approach developed by Kasianchuk at the University of California (Berkeley) may be more appropriate. Thus, the critical or effective AC modulus E_1 or design stiffness, S_d would be:

$$S_d = \sum_{i=1}^{x} \frac{(S_{mix} \text{ at } T_i)(N_i)}{N}$$

where a given pavement temperature interval T_i has N_i traffic repetitions associated with it. At the mean pavement temperature within any given interval an associated mix stiffness S_{mix} can be determined from direct testing or nomographic solutions.

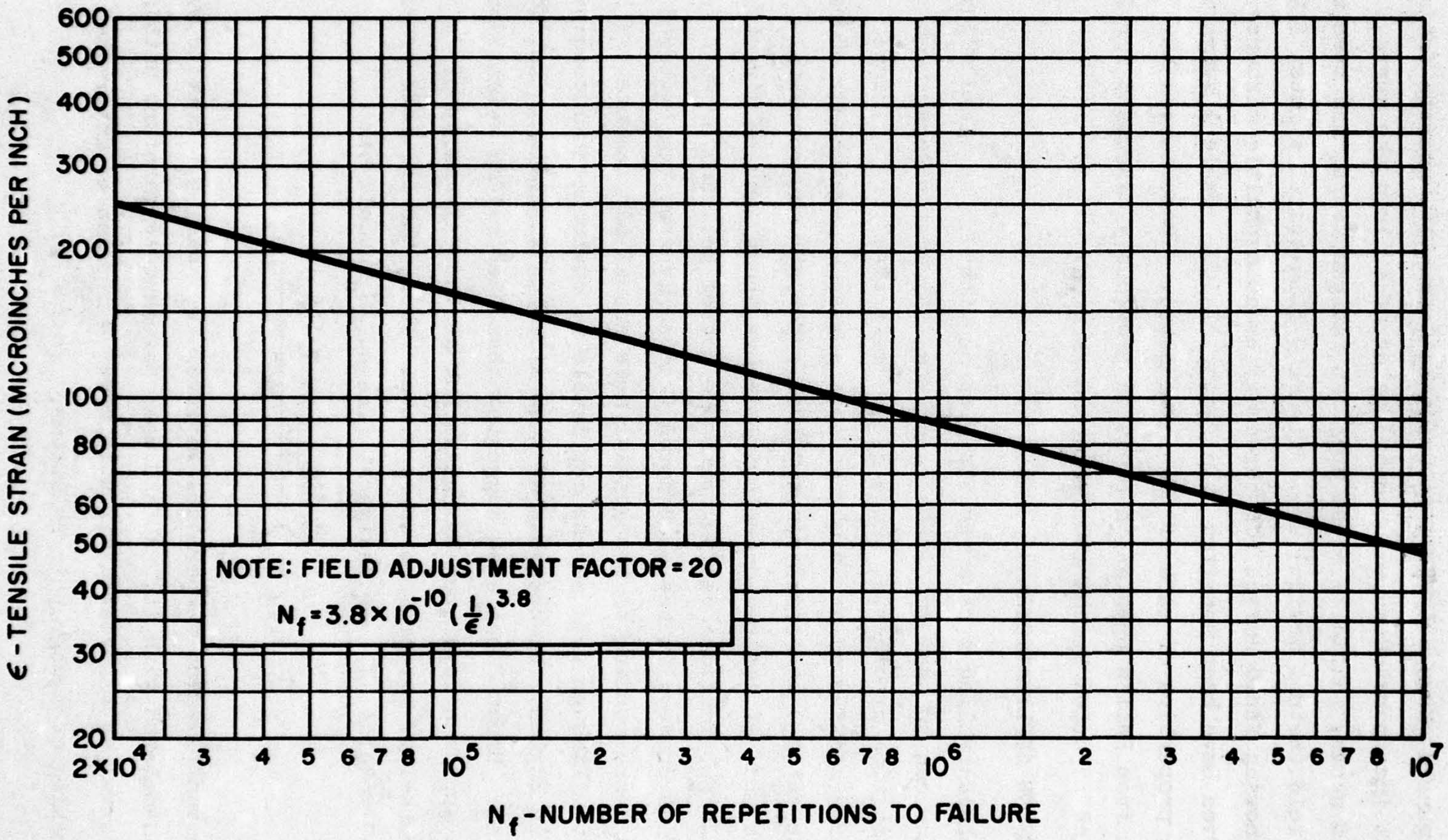


Figure 8. Typical asphalt concrete fatigue curve adjusted for in situ conditions (from Brown and Pell[2])

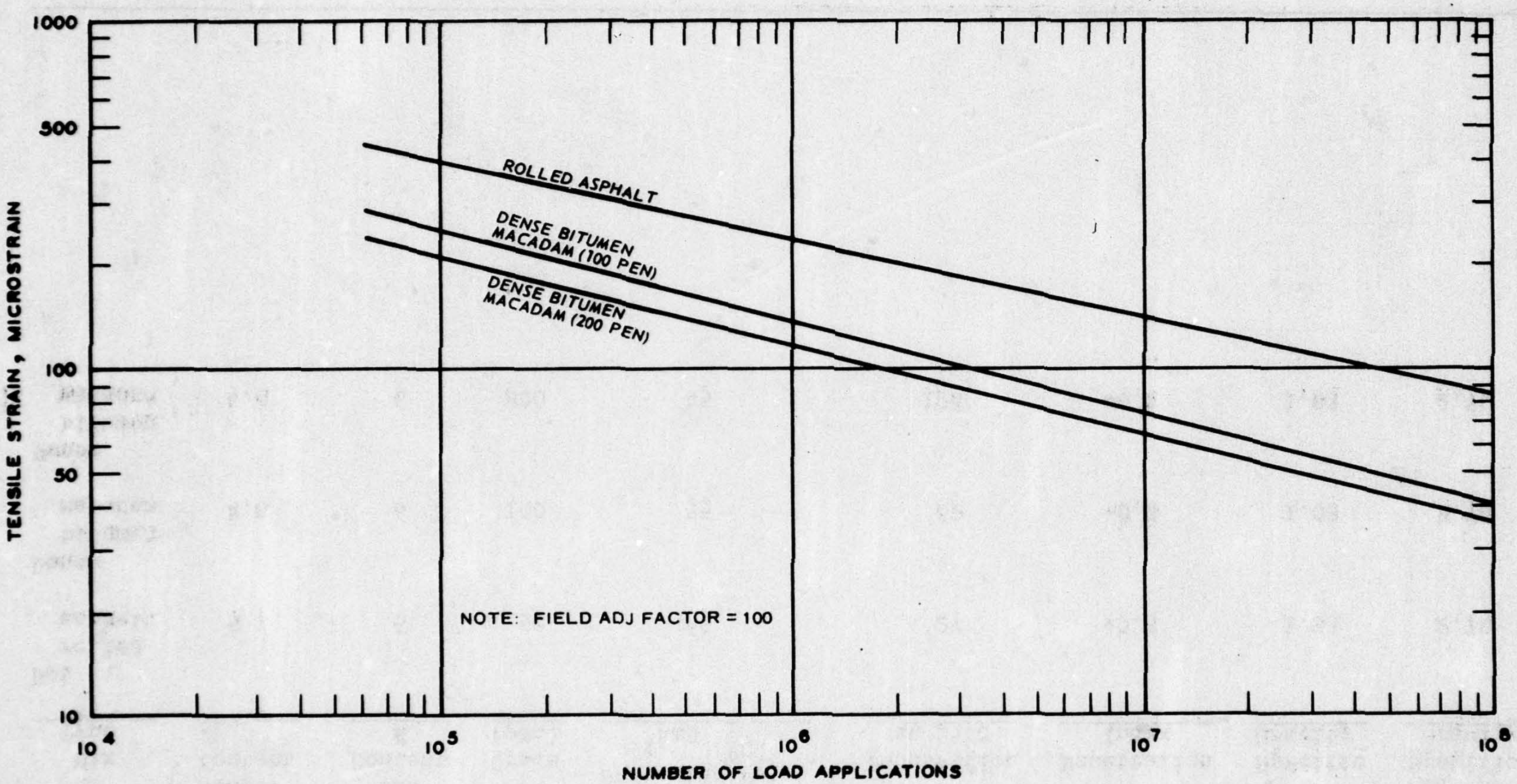


Figure 9. Approximate fatigue lines for typical bituminous mixes adjusted for in situ conditions (from Brown[11])

Table 1

Typical Mix Properties for Fatigue Results

Mix Type	Binder Content %	Void Content %	Binder Properties						Aggregate Specific Gravity
			Grade (pen)	Softening Point $T_{R\&B}$ (°C)	Penetration at 25°C	Penetration Index	Specific Gravity		
Hot rolled asphalt	5.7	6	50	65	27	+0.3	1.03		2.70
Dense bitumen macadam	4.0	6	100	55	62	+0.2	1.03		2.70
Dense bitumen macadam	4.0	6	200	45	126	-0.4	1.03		2.70

UNIVERSITY OF CALIFORNIA
(BERKELEY)

Background. The investigations concerning asphaltic material research at the University of California (Berkeley) have been conducted under the direct supervision of C. L. Monismith. The major areas of research endeavor have focused upon: (a) the development of laboratory fatigue testing and the effect of mix properties upon flexural behavior,[14-21] (b) the relationship between tensile properties and distress (fracture) manifestations,[20-24] and (c) the development of a structural fatigue subsystem to include verification studies with prototype slabs and actual field pavements.[17,25,27-29] Additionally, fatigue behavior of cement-treated material has also been conducted and the results are discussed subsequently.[30]

This concerted research effort was initiated in the early 60's and has been continually progressing. Unlike Pell's efforts at Nottingham, it appears that the studies at Berkeley have focused upon a more totally implementable fatigue design system. Nonetheless, a great deal of effort regarding the laboratory behavior of asphaltic mixtures and their subsequent dependence upon mix properties has been conducted with numerous types of mixes and many hundreds of individual fatigue tests.

Another marked distinction between the Berkeley studies and those at Nottingham concerns the differences in test apparatus used to monitor the fatigue behavior. At Berkeley, use has been made of repeated flexural testing on rectangular beam specimens. Figure 10 shows the types of apparatus used. The controlled strain fatigue test device is illustrated in diagram a, while diagram b illustrates the apparatus developed by Deacon and used primarily for controlled stress tests.

In the controlled strain device, the beam specimen, shown as 2 in. deep by 3 in. wide by 12 in. long, is loaded in simple bending. The load is applied through an air cylinder acting against a loading yoke while the specimen rests in a spring foundation.[16] Two 1-in. strain gages were bonded to each specimen and monitored continuously throughout the test to allow constant strain conditions. Load rate (frequency and load duration) were monitored electronically to allow a

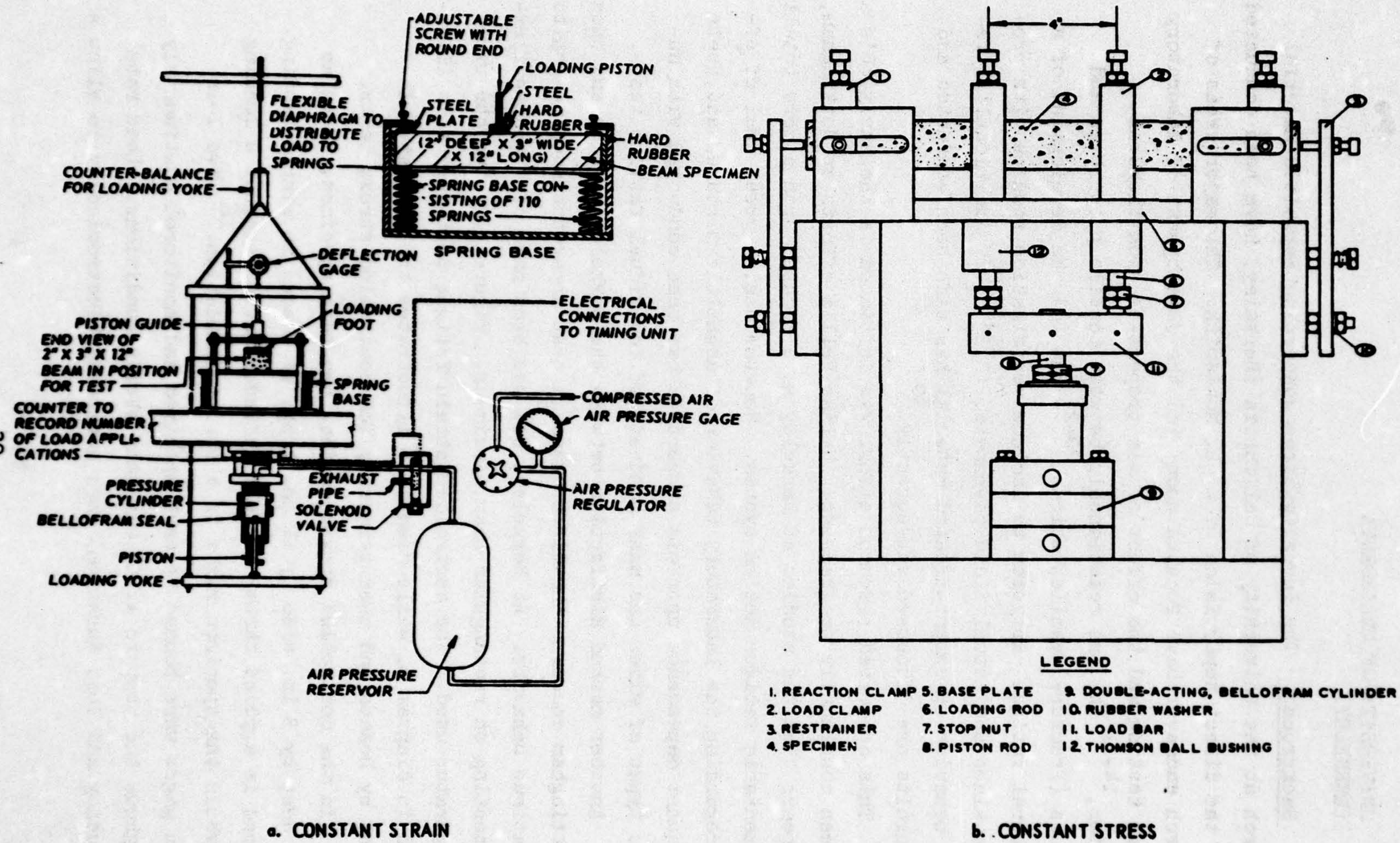


Figure 10. Repeated flexural apparatus for constant strain and constant stress testing (from Monismith[16])

wide range of variables to be considered. Dynamic stiffnesses (moduli) were calculated directly from elastic beam equations by knowledge of the strain and stress (load) after any number of test repetitions. The latter point (stiffness) determination directly from the fatigue measurement phase is another feature which differed from Pell's study at Nottingham.

The controlled stress device, developed by Deacon, can also be used for controlled strain testing. In the former test procedure, load clamps are required to allow for load reversal necessary to force the bituminous specimen back to its original undeflected position. For controlled stress conditions, generally dynamic deflections, measured with linear variable transducers (LVDT) are recorded on strip charts, and used to determine the flexural stiffness of the specimen during the test. The 1-1/2- by 1-1/2- by 12-in. beam specimen is loaded in third-point loading so that a constant moment (stress) exists within the central portion of the specimen. The stiffness S is obtained by:

$$S = K\left(\frac{P}{I\Delta}\right)$$

where

K = a loading geometry constant

P = the applied load

I = the moment of inertia of the beam

Δ = the measured dynamic deflection of the beam center

Thus for both apparatus, stiffness measurements are intrinsic to the parameters necessary to conduct the fatigue test. This allows for a specific stiffness value made on the exact beam specimen being fatigued at the same temperature and frequency of load rate.

Major Findings. Monismith and associates have conducted a rather extensive study relative to the effects of mix properties and test conditions upon fatigue behavior of bituminous mixes. The primary mixture factors that were investigated were mix stiffness, air voids, aggregate gradation, aggregate type, asphalt type, and asphalt content. Although much of the work included laboratory prepared specimens, fatigue tests were also conducted using beam specimens sawed from in situ pavements.

It is very important for the reader to note that the majority of the initial fatigue work was accomplished only at one temperature condition, namely 68°F. The following discussion relative to the effect of mix variables upon fatigue is thus based upon stiffness differences solely due to mixture differences at this test temperature and conducted under controlled stress conditions.

Like many other researchers, Monismith found the effect of air voids upon fatigue performance to be a very significant variable. Figure 11 illustrates the results obtained by Monismith for two different

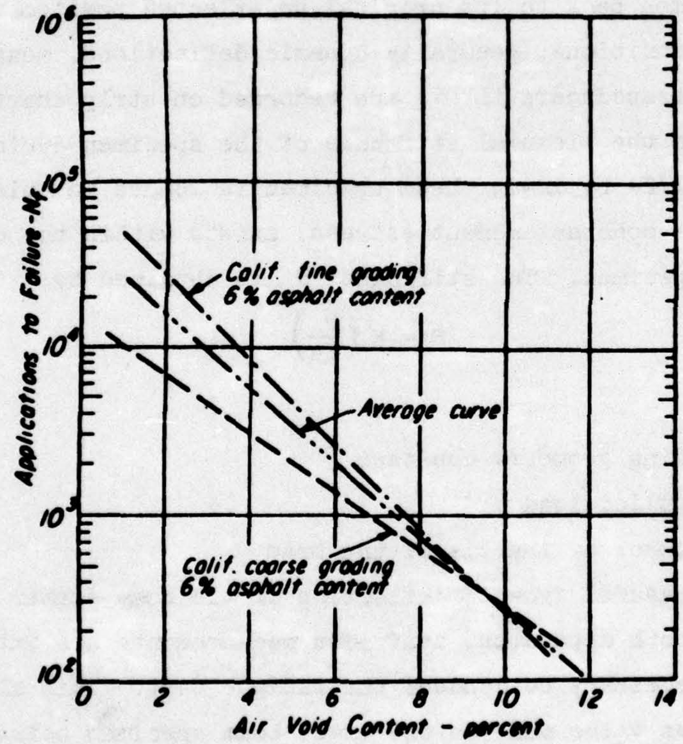


Figure 11. Influence of air void content upon fatigue performance (from Monismith et al.[14])

aggregate gradings. As can be seen the effect of increasing the void content significantly reduces the fatigue performance at a given stress level. Comparing these results with the results of other fatigue investigators, Monismith concluded that other void characteristics, such as size and shape, may also affect fatigue performance rather than air void content.

For the given test temperature, the mix stiffness was found to primarily affect the slope of the strain-life relationship. This is illustrated by Figure 12 which shows that mixes with higher stiffnesses

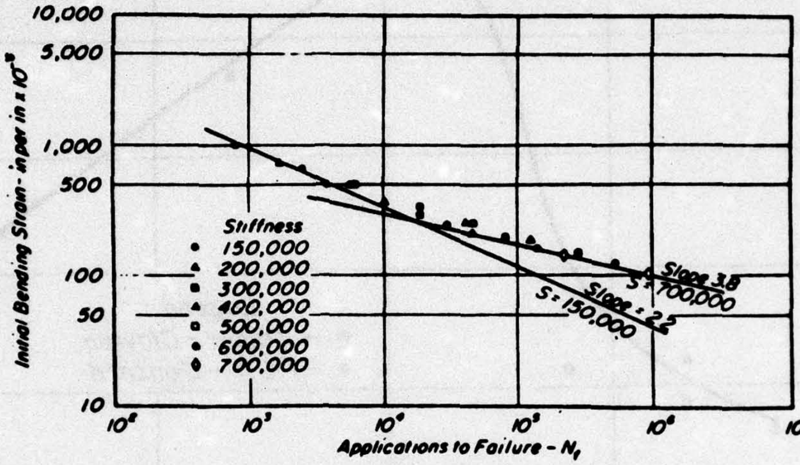


Figure 12. Effect of mix stiffness upon fatigue performance--68°F (from Monismith et al.[14])

result in flatter slopes (larger values of m) than mixes with lower stiffnesses. The reason for this, suggested by Monismith, is related to the differences in crack propagation times associated with the different mix stiffnesses and necessary stress values required to fatigue the specimens.

Similar to the observations made at Nottingham, the effect of both aggregate type and grading appeared to have very little if any effect upon the strain-life relationship obtained. It was also found that the asphalt type played only a minor role in affecting the strain-life relationship for the relatively small range in mix stiffnesses investigated. It was also concluded that, for maximum fatigue life, an optimum asphalt content existed that nearly equals a condition of maximum mixture stiffness. This result is shown in Figure 13.

In addition to the investigation of various mix properties, Monismith also studied a similar mix that Pell, in his Nottingham study, also researched (British Standard No. 594, Gap Graded). Although different crushed aggregate types were used between the two investigators, it was found that different strain-life relationships were obtained.

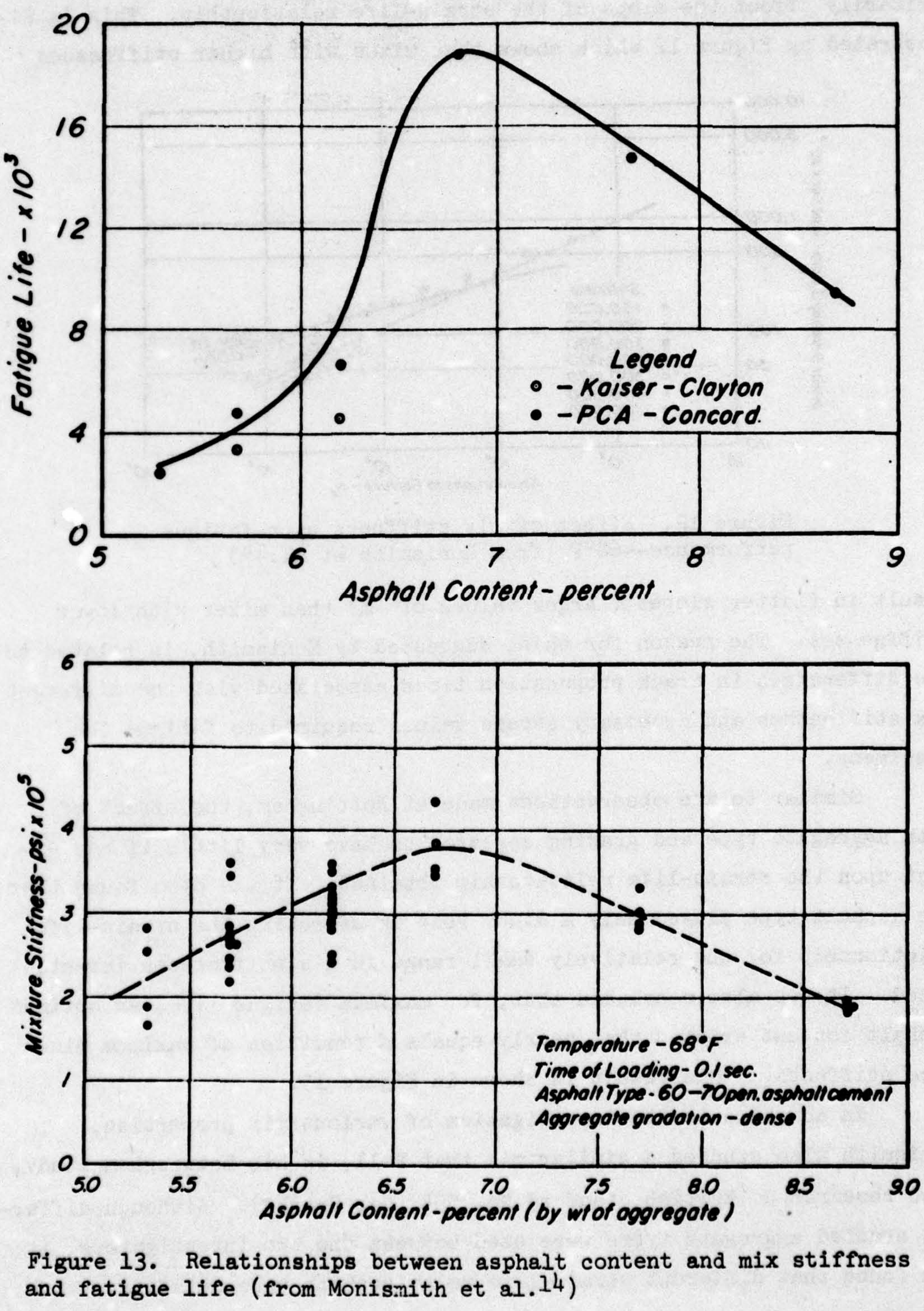


Figure 13. Relationships between asphalt content and mix stiffness and fatigue life (from Monismith et al.[14])

Pell's research indicated a much steeper (larger in values, i.e., m = 6.1) slope compared with the California study (m = 3.4). It was suggested that possible reasons for this discrepancy were due to: (a) difference in fatigue testing apparatus, (b) difference in methods of statistically regressing (selecting the independent variable) the results, (c) differences associated with the determination of the mixture stiffness, and (d) the differences in the test temperature(s) used by both investigators.

However, Monismith, like Pell, also found that a relationship between the fatigue equation intercept and slope factor (power) did exist for the various mixes studied. This comparison is illustrated by Figure 14 and does show remarkably good correlation considering the differences in test procedures and specimens investigated by the two researchers.

Although it has been previously noted that in the study involving

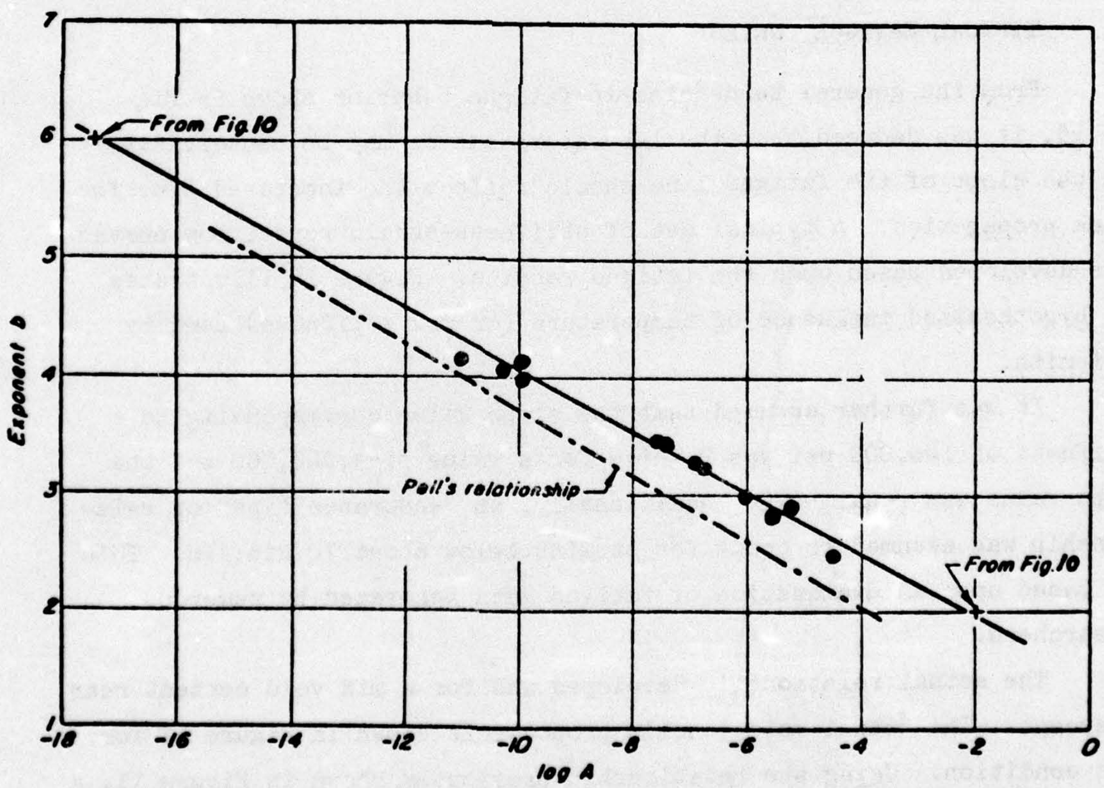


Figure 14. Relationship between log A and b in equation $N_f = A(1/\varepsilon)^b$ (from Monismith[16]) where N_f = repetitions to failure and ε = strain magnitude

the effect of mix properties upon fatigue performance only one test temperature (68°F) was used; other studies on specific specimens were conducted at other temperature conditions. Figure 15 illustrates the results on two different mixtures for different test temperatures. It is readily apparent that unique strain-life relationships are noted for each temperature even when stiffness is accounted for by plotting the results as a function of initial mix strain. This is in direct contrast to the "strain criterion" proposed by Pell. However, it should be recalled that Pell did point out that this possibility may exist at elevated temperatures.

Based upon these total findings it was concluded by Monismith that the two most influential parameters affecting the strain-life relationship (at least for mixes typical to California specifications) were the mix stiffness and air void content.

TYPICAL FATIGUE CURVES

From the general tendencies in fatigue behavior shown in Figure 15, it was deduced that the laboratory curves may be conservative and the slope of the fatigue line should reflect the increased time for crack propagation. A typical set of stiffness-strain repetition curves were developed based upon the fatigue results. Figure 16 illustrates the hypothesized influence of temperature (or mix stiffness) used by Monismith.

It was further assumed that the slope value corresponding to a stiffness of 100,000 psi was 2 while for a value of 4,000,000 psi the slope value was equal to 6. Additionally, an "endurance type" of relationship was assumed to occur for strains below about 70 μin./in. This was based upon an examination of fatigue data generated by numerous researchers.

The actual relationship developed was for a mix void content near 7 percent. The actual relationship proposed is shown in Figure 17 for this condition. Using the relationship previously shown in Figure 11, a more universally applicable fatigue relationship was developed for a

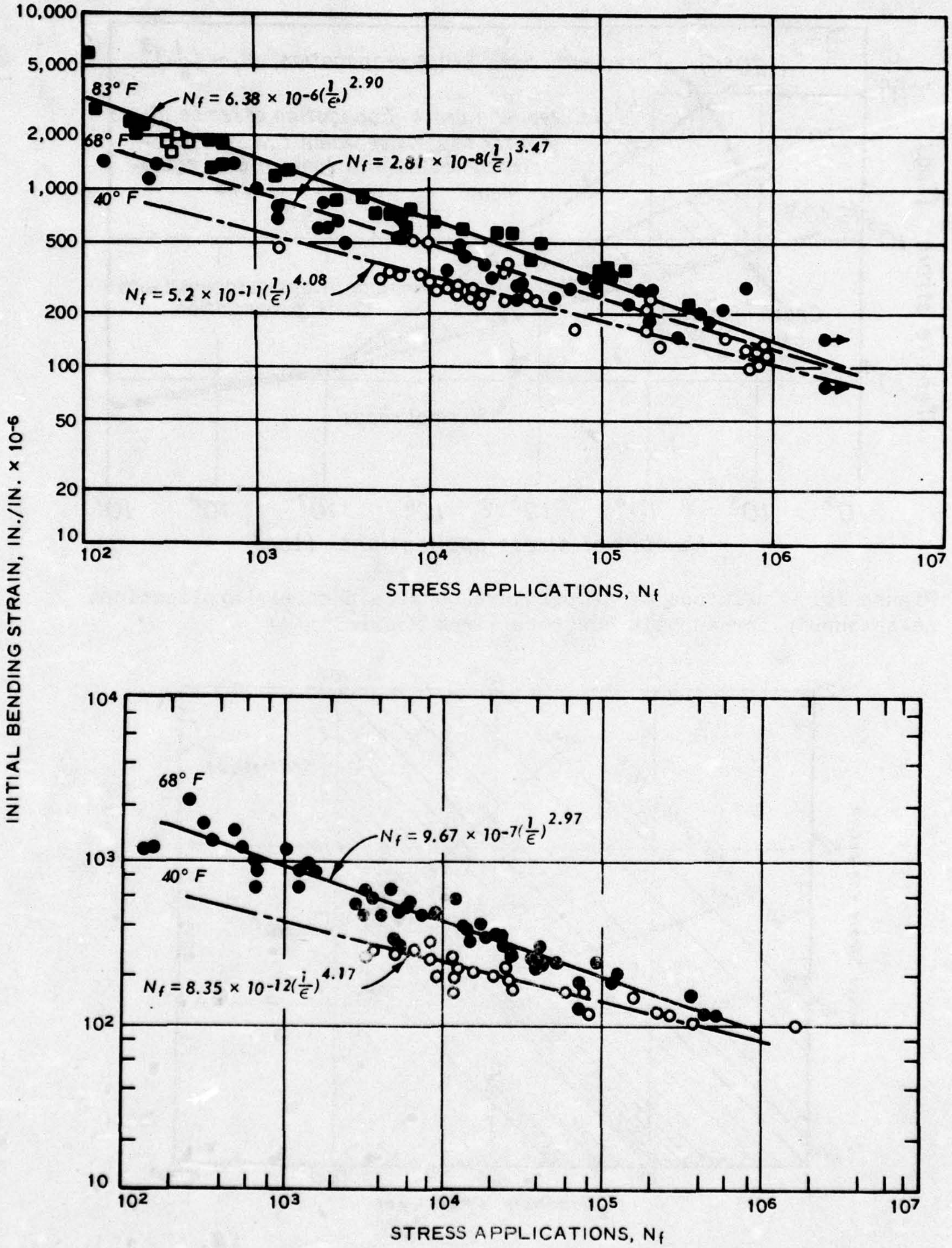


Figure 15. Influence of test temperature upon fatigue performance for two different mixes (from Monismith et al.[14])

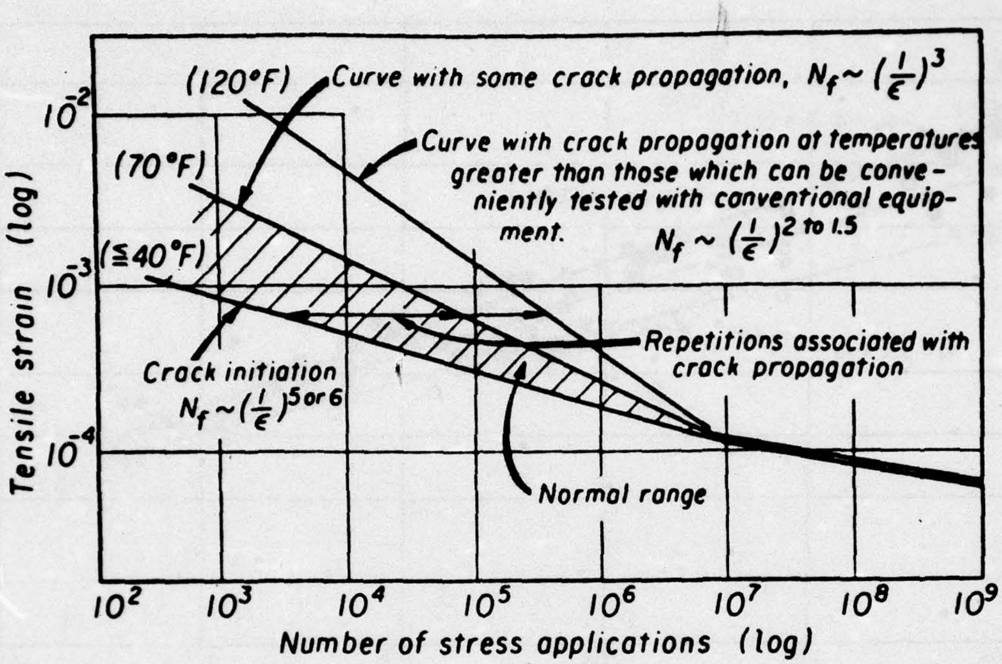


Figure 16. Influence of temperature on strain-stress applications relationship for asphalt concrete (from Monismith[16])

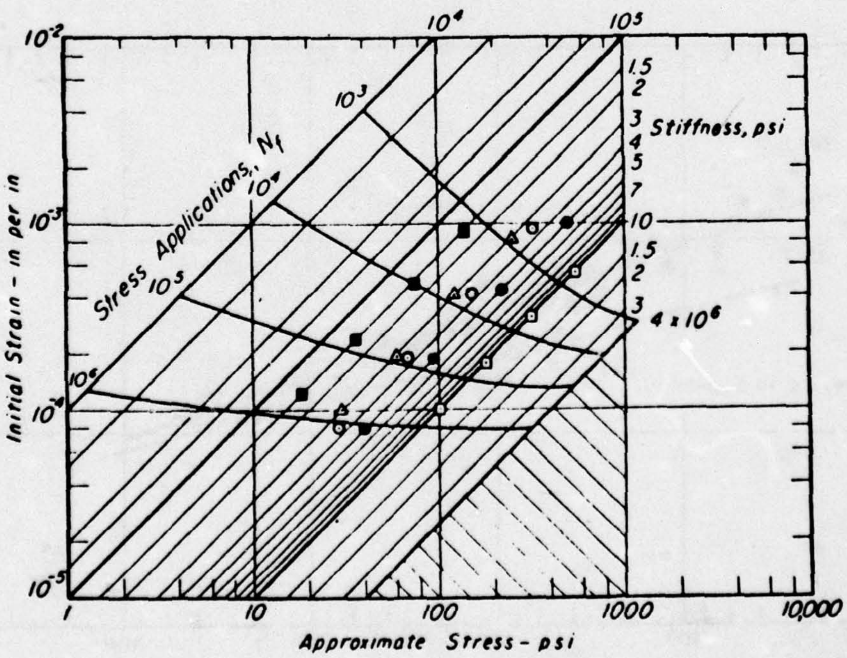


Figure 17. Fatigue criterion for California-type mixes--mix void content = 7 percent (from Monismith[16])

void content of 5 percent. The suggested typical fatigue relationship is shown in Figures 18 and 19.

DESIGN APPLICATIONS AND VERIFICATION

The use of fatigue test results within a structural fatigue distress subsystem has been developed by Monismith and associates.[17,19,20,25-27] Early publications appear to have been influenced by the use of only one test temperature (e.g., 68°F) to evaluate the constant stress or strain fatigue behavior and use these results within an elastic layered model to compute the layered pavement strains. Later publications, however, refer to the adoption of the stiffness, strain, and life relationship shown in Figure 19.

In the developmental work, use of the Mode Factor to precisely define the intermediate fatigue mode (between the limits of constant stress and constant strain) has been advocated. Additionally with the use of a single or unique fatigue curve, the concept of a mean weighted traffic stiffness approach has also been suggested to accumulate fatigue

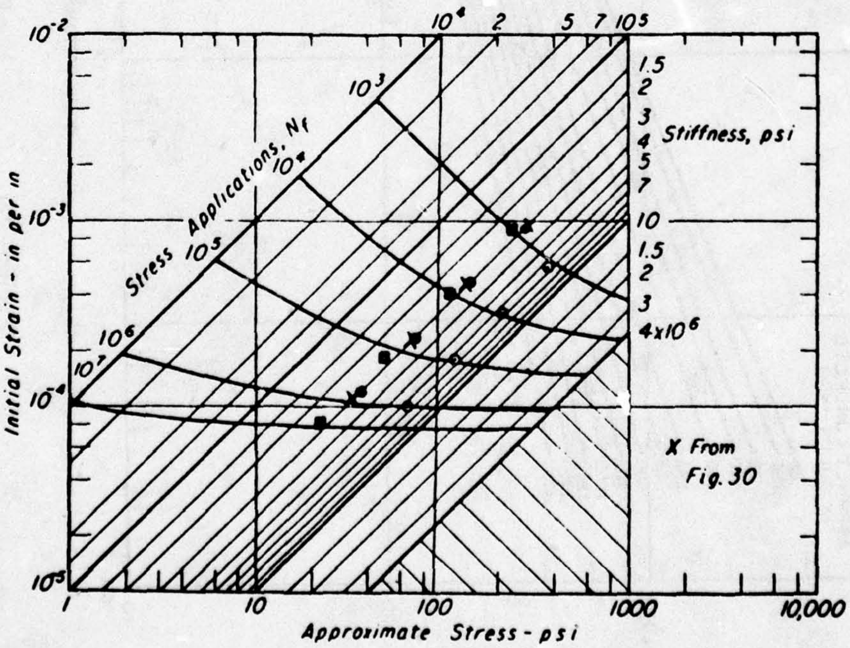


Figure 18. Fatigue criterion for California-type mixes--mix void content = 5 percent (from Monismith[16])

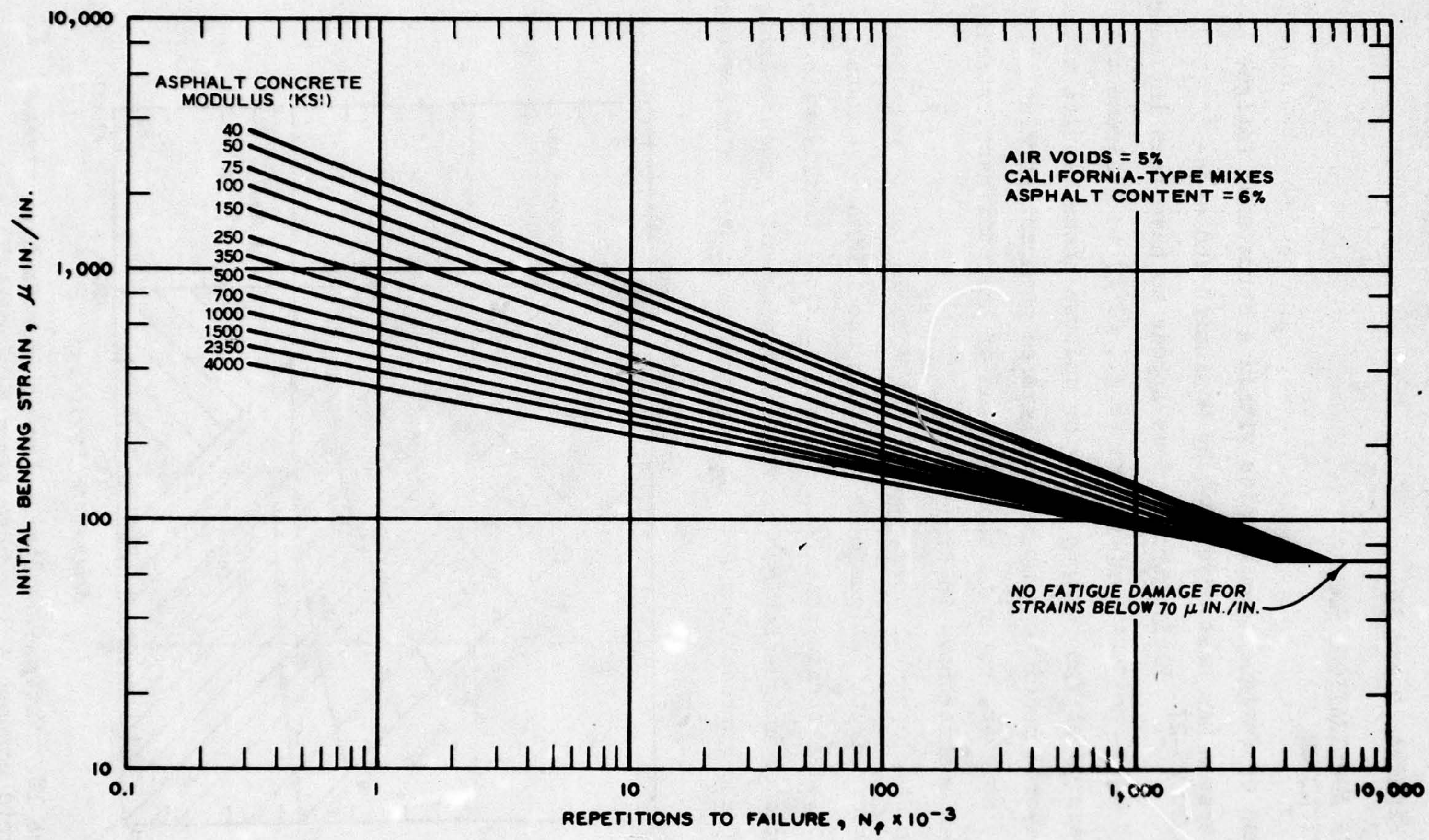


Figure 19. C. L. Monismith fatigue criterion[16]

damage for the expected range in field mix stiffnesses. However, with the curves shown in Figure 19, it is quite direct to use cumulative damage studies to check or design pavements for fatigue distress.

Several of the concepts presented have been verified by either prototype laboratory slab studies or an analysis of in situ pavement performance. The initial type verification study was made on the Morro Bay Project.[20] In this analysis a controlled strain test defined at 50 percent stiffness reduction at 68°F was found to be appropriate for the investigation. Using elastic layered theory and dynamic load tests to characterize the other unbound pavement layers, predicted lives of 0.8 yr at a 90 percent confidence level and 1.8 yr at a mean fatigue relationship were found to exist. Subsequent studies involving the cutting of rectangular slabs from the pavement resulted in the existence of crack patterns at the bottom of the layer (not being propagated to the surface at the time of the study) some 3 yr and 10 months after the pavement was opened to traffic.

More recently, Yuce and Monismith[28] investigated the prediction of fatigue cracking using controlled stress fatigue results (68°F) with observed crack initiation on prototype slabs subjected to repeated loads. In the study, crack initiation was monitored by crack sensors (silver conductance paint and/or aluminum foil tape) and by measured slab deflections. Beam specimens were cut from the slabs for laboratory fatigue tests. The comparison between observed lines from deflection and crack sensing devices and service lives predicted from repeated flexural fatigue tests is shown in Figure 20. The obviously good agreement between the procedures can be observed.

Finally, the author has conducted a study based upon a comparison of predicted fatigue behavior and actual performance at Baltimore-Washington International Airport.[31] Using the criterion shown in Figure 19, predicted repetitions to fatigue failure for a particular taxiway section were between 6000 and 7000 DC-8-63F strain repetitions. A cumulative fatigue damage model and conjunctional dynamic load tests (lab) on the subgrade soil were used in the prediction. Analysis

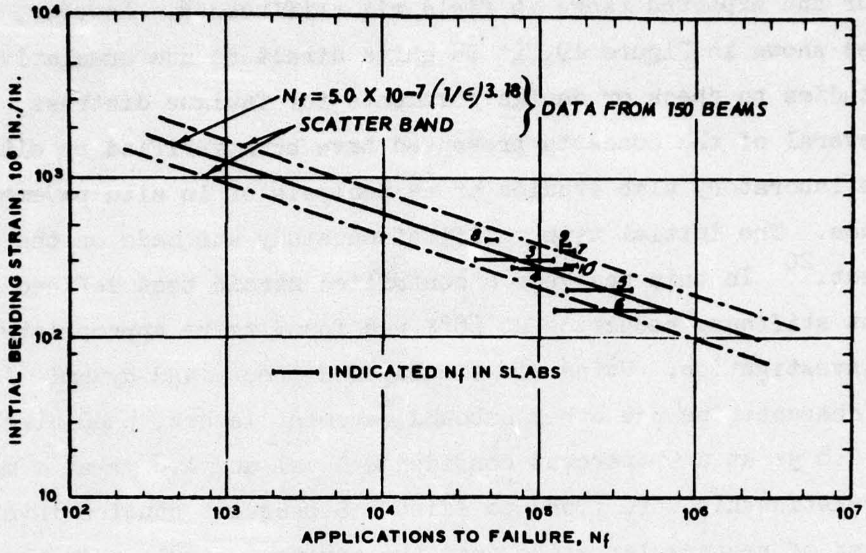


Figure 20. Comparison of service lives obtained from beam and slab tests (from Yuce and Monismith[28])

of the actual taxiway performance showed that about 9,000 strain repetitions occurred before the onset of surface cracking while about 13,500 strain repetitions occurred before a "functionally" failed condition resulted. Hence it would appear that even though a limited number of verification studies have been made, some credence to the direct use of the fatigue subsystem recommended can be given.

CENTRE DE RECHERCHES ROUTIERES--BRUXELLES (CENTER OF ROAD RESEARCH--BRUSSELS)

Background. Extensive fatigue testing of bituminous mixtures has been conducted by J. Verstraeten of the Road Research Center in Brussels over the past several years. The major objective of Verstraeten's work has focused on defining the effect of mix properties upon fatigue performance.[32-34] Controlled stress laboratory testing has primarily been the major source of research information. In his study, Verstraeten has investigated the fatigue behavior of over 40 different mixes and has studied fatigue response over temperature and frequency ranges of -20°C to +30°C (≈ 0°F to 88°F) and 3 Hz to 100 Hz, respectively.

Unlike the test specimens of Pell, Monismith, Kirk, and others,

Verstraeten used specimens that were trapezoidal in shape with bases of 9 cm and 3 cm; height was 35 cm and thickness 3 cm. Specimens were obtained by sawing them to the desired shape from large rectangular blocks. During testing the large dimension base was fixed while the other base was exposed to a sinusoidal continuous load. Stiffnesses were directly determined on the specimens during the fatigue test. However, it has been noted by Verstraeten that..."moduli are determined for a stress such that if it is repeated about 10^7 times or more, fatigue failure occurs." This implies that a flexural stress of small magnitude was used in the stiffness calculations and therefore appears that direct non-linear effects of moduli may not necessarily be accounted for in the determination of the tensile strain.

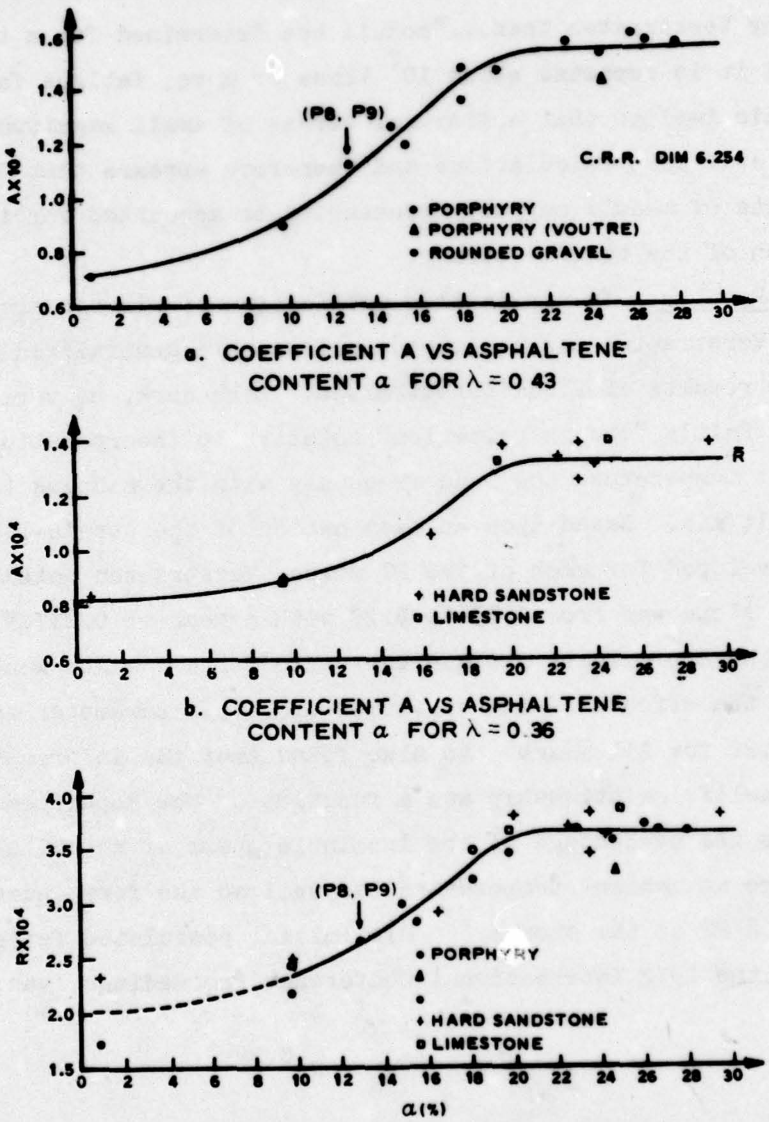
Test Results. In his initial publication of his research findings,[32] Verstraeten based his conclusions on a generalized fatigue law upon the results of 27 different mixes. Like Kirk, he verified the existence of Pell's "strain criterion" relative to incorporating the effects of test temperature and load frequency with the modulus (stiffness) of the asphalt mix. Based upon an examination of the strain-life relationships developed for each of the 27 mixes, Verstraeten noted that the range in the slope was from 0.19 to 0.27 with a mean of 0.217.* He concluded that the approximate fatigue law valid for all mixes would be independent of the effects of mix variables upon this parameter and adopted a value of 0.22 for all mixes. He also found that the intercept value of the strain-life relationship was a function of the asphaltene content α (defined as the percentage of the insoluble phase of the bitumen in normal heptane at ambient temperature) as well as the form index λ of the fraction 8/22 of the stones.[32] His initial postulated fatigue law, presented in the 1972 International Conference Proceedings, was:

$$\varepsilon_N = \overline{R} \cdot \lambda \cdot V_B \cdot N_f^{-0.22}$$

* Note that Verstraeten's slope is the reciprocal of most other investigations.

where $\overline{R}$ is the coefficient shown in Figure 21c and functionally related to the asphaltene content α , the coefficient λ is as defined previously, and V_B is the percentage volume of the binder (relative to <u>total</u> volume including voids).

In his 1972 prepared discussion of his original Proceedings



c. COEFFICIENT R VS ASPHALTENE CONTENT α FOR ALL THE MIXES

Figure 21. Verstraeten's asphaltene content factor[32]

paper,[33] Verstraeten modified the original fatigue to incorporate the effects of seven additional mix types. His new law was stated as:

$$\varepsilon_N = \overline{\phi} \cdot C \cdot \left(\frac{V_B}{V_B + V_V}\right) \cdot N_f^{-0.22}$$

The coefficient $\overline{\phi}$ was related to the asphaltene content, and its value can be seen in Figure 22. Figure 23 illustrates the C correction factor which is functionally related to the ratio of aggregate volume to the volume of voids $(V_B + V_V)$ in the mineral aggregate.

His most recent publication[34] has resulted in another minor modification to reflect research findings from 42 mixes. The latest

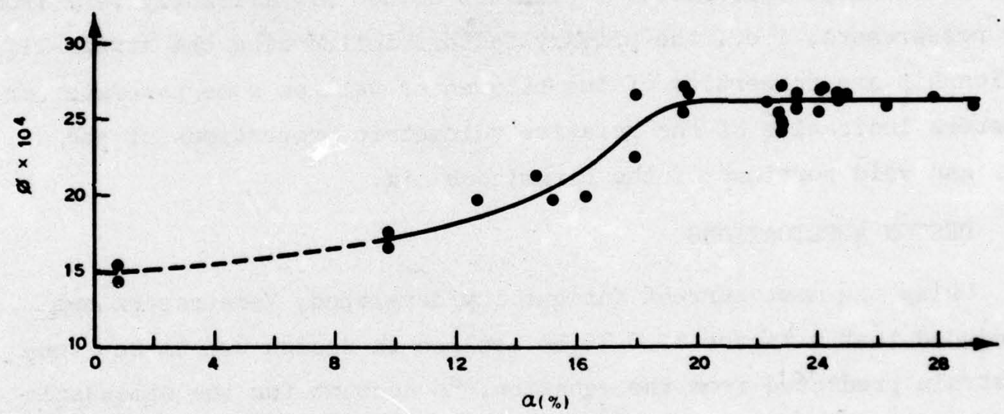
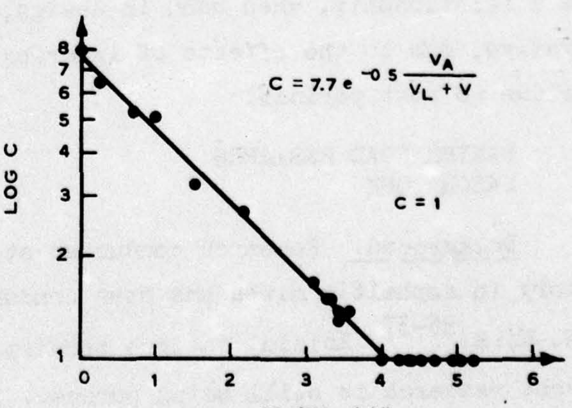


Figure 22. Variation of coefficient $\overline{\phi}$ in function of the asphaltene content α in the bitumen

Figure 23. Variation of coefficient C in function of ratio $V_A/(V_L + V)$



fatigue law advocated by Verstraeten is:

$$\epsilon_N = \overline{\Lambda} \left(\frac{V_B}{V_B + V_V} \right)^{-0.21}$$

This latter equation is quite direct in its use as the $\overline{\Lambda}$ value is determined directly from Figure 24 as a function of the asphaltene content of the bitumen. Verstraeten has also published general approximations of the $\overline{\Lambda}$ function relative to ring and ball temperature, viscosity, and viscosity ratios in lieu of the asphaltene content variable.[34] Figure 25 illustrates such a correlation to the more widely used ring and ball temperature parameter.

In summary, Verstraeten's findings do not significantly vary from other researchers; i.e., the primary factors influencing the strain-life relationship are properties of the bitumen as well as some parameter or parameters indicative of the relative volumetric proportions of the binder and void portions of the bituminous mix.

DESIGN APPLICATIONS

Using the most current fatigue law developed, Verstraeten has recommended that a factor of 0.75 be applied in design use to the tensile strain predicted from the equation, to account for the stochastic nature of the fatigue results.[32] This corresponds to a strain-life relationship in which only about 2.5 percent of the results would be more severe than the design relationship. Acknowledgement is made that such a relationship, when used in design, would tend to be quite conservative, due to the effects of ignoring the beneficiation in fatigue life due to rest periods.

DANISH ROAD RESEARCH
LABORATORY

Background. Research conducted at the Danish Road Research Laboratory in asphaltic mixes has been conducted under the auspices of J. M. Kirk.[35-37] Initial fatigue studies started in the mid-60's and current research is still being pursued. Kirk's work, like that of Pell, Monismith, and Verstraeten, has been principally concerned with the

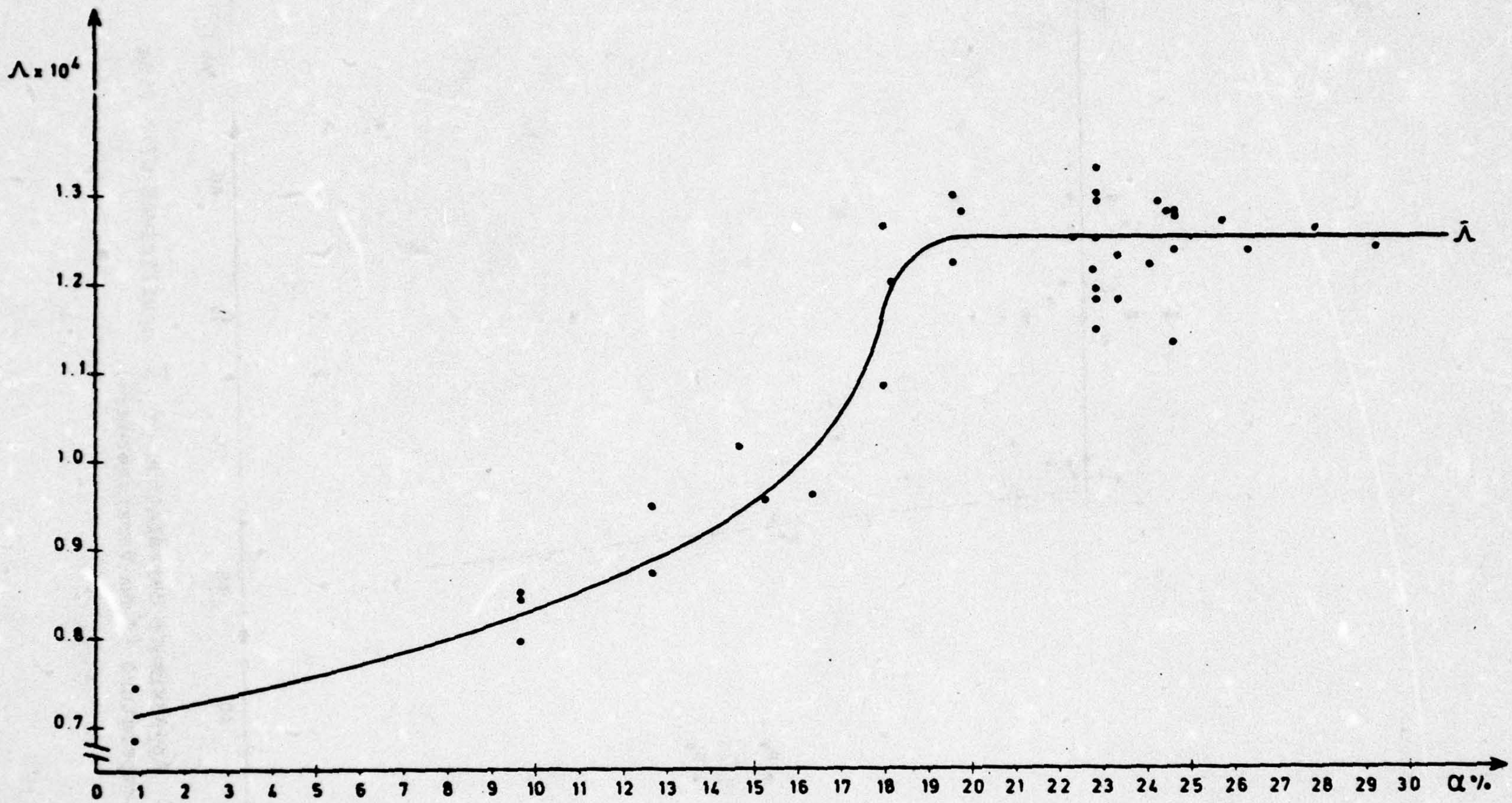


Figure 24. Variation in the coefficient $\bar{\Lambda}$ as a function of the asphaltene content α (from Verstraeten[32])

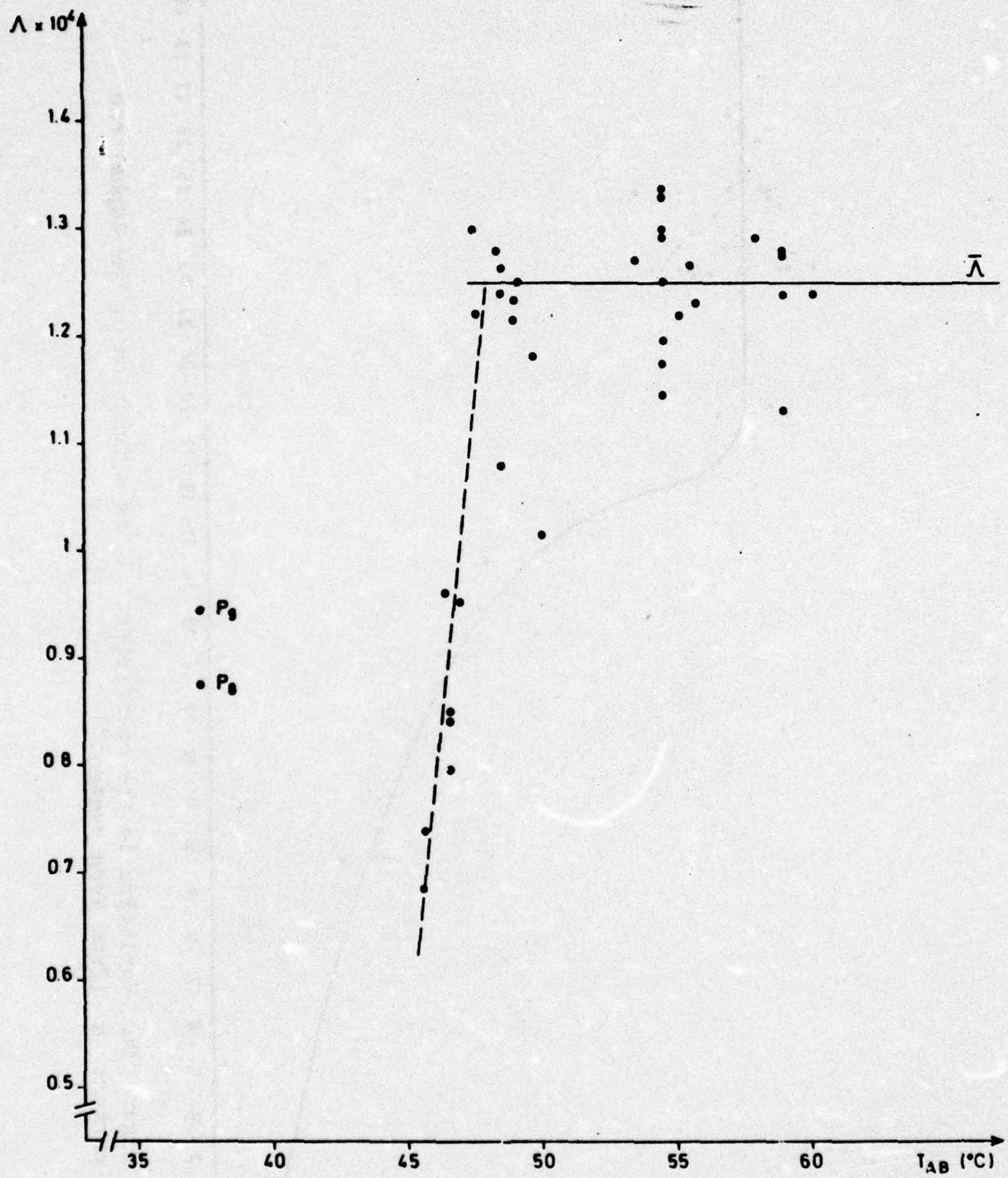


Figure 25. Approximate correlation of $\bar{\Lambda}$ coefficient with ring and ball temperature (from Verstraeten[32])

influence of mix properties upon laboratory fatigue behavior. Almost all results published have been applicable for constant-stress-type testing. Very little, if any, work has been found in the literature applying Kirk's results within a structural fatigue design subsystem.

The test apparatus used by Kirk is quite similar to that used by Monismith in that rectangular beam specimens 5 cm by 7 cm by 35 cm (2 in. by 2-3/4 in. by 13-3/4 in.) are used in third-point repeated flexural loading.[35] A continuous sinusoidal loading has been primarily used during the testing phase with temperature ranges of 0°F to about 85°F being considered for some tests. However, as will be noted later, limited results using an impulse load (half sine wave load 0.02 sec with 0.5 sec dwell time) have been presented.[35] It also appears that direct stiffness determinations have not been made on test specimens, as reliance has been with the Shell stiffness nomograph devised by Van der Poel.[38] This point, however, is not absolutely confirmed in Kirk's publications.

Test Results. The majority of Kirk's work has concentrated on continuously graded mixes, although some research on the British gap graded mix has been done. From the results of continuous sinusoidal testing, Kirk has concluded that the effects of temperature, rate of loading, and penetration grade of the bitumen can be accounted for, in a given mix, by the concept of a single parameter, the mix stiffness. Thus, the validity of Pell's "strain criterion" was likewise shown of Kirk's results.

Relative to the mix properties it was found that the strain required to reach a given fracture life (e.g., $N_f = 10^6$) increased with increasing bitumen content. Kirk also deduced that the air voids must also have an effect upon fatigue strength. However, studies showed that the effects of both of these variables could be incorporated by looking at the fatigue strain per unit volume of binder $\varepsilon \times 10^6/V_B$ and the volumetric ratio of binder to air voids V_a. This is shown in Figure 26. For a V_B/V_a ratio of less than about 5 it can be seen that the strain per unit binder volume increases with decreasing V_B/V_a ratios. However, for values greater than this (≥ 5) the air voids have no effect upon the strain-life relationship. Figure 27[36] is a more recent plot showing

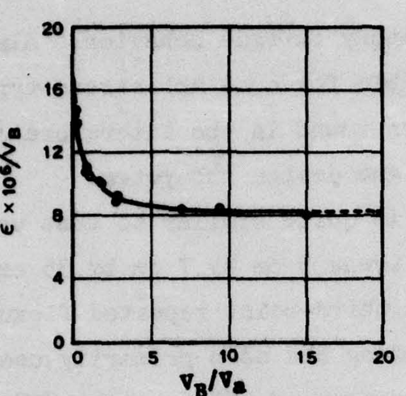


Figure 26. Fatigue strain per volume percent binder plotted against binder-air volume ratio

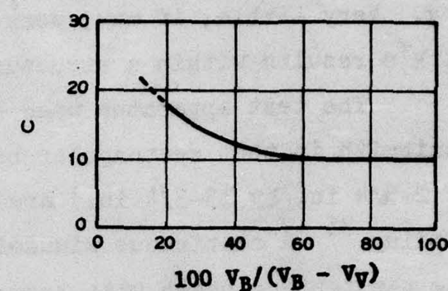


Figure 27. Correction factor C versus percentage of voids in mineral aggregate filled with bitumen

a similar trend. In lieu of using the V_B/V_a ratio, the binder volume to voids in the mineral aggregate ratio is plotted against a correction factor C . The use of this parameter will be explained in the next few paragraphs.

Kirk initially found that for the aggregate type and grading effect upon fatigue performance only a negligible difference in behavior occurred.[35] However, in later publications the effects of maximum aggregate size and filler content were stated to be other primary factors influencing the strain-life relationship.[36-37] Figure 28 illustrates the effect of increasing maximum aggregate size upon strain per unit volume of binder to achieve a given fracture life of 10^6 repetitions. True quantitative effects of filler content have not, as yet, been formulated by Kirk relative to fatigue behavior. A provisional type of rule formulated by Kirk is that the percentage (by weight) of filler material (passing No. 200 sieve) should be approximately equal to the binder content percentage (by weight) in a given mix. If this filler percentage is less than this value the

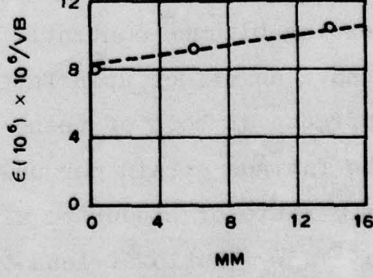


Figure 28. Strain per volume percent binder versus maximum size of aggregate

fatigue life is decreased rather sharply.

<u>Typical Fatigue Curves.</u> Based upon the laboratory results, Kirk has postulated a generalized fatigue prediction equation.[37] It is:

$$\varepsilon_N = \frac{N_f^{-a}}{10^6} \times \frac{\varepsilon \times 10^6}{V_B} \times V_B \times C_1 \times C_2 \times C_3 \times C_4 \times C_5$$

where

ε_N = initial strain at N repetitions

N_f = repetitions to fracture

a = the slope of the fatigue curve

$(\varepsilon \times 10^6/V_B)$ = the strain per unit volume of binder

V_B = the percent binder volume

C_1 to C_5 = correction factors

In the expression above, the value of a has been found to be a function of the binder stiffness. The value can be obtained directly from Figure 29 by taking the vertical distance between the two fracture levels shown on the solid curves for a particular binder stiffness. Likewise the strain per unit volume value at 10^6 repetitions can also be read off Figure 29 at the bitumen stiffness.

The correction factors shown in the equation account for: (a) maximum aggregate size--C_1, (b) percent of voids in the mineral aggregate filled with bitumen--C_2, (c) filler to bitumen ratio--C_3, (d) asphaltene content correction--C_4, and (e) rest period and temperature effects--C_5. Figure 28, previously shown, illustrates the C correction factor for aggregate size. Figure 27 illustrates the C_2 factor for percentage of voids in the mineral aggregate filled with bitumen. The magnitude of C_3 is not clearly defined except by using a value of 1.0 for filler to bitumen weight percentage ratios of unity. As a guide,

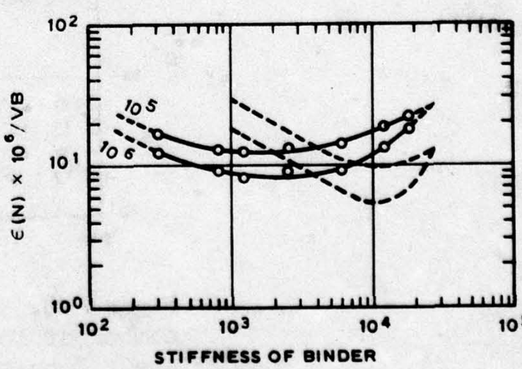


Figure 29. Strain per volume percent binder versus stiffness of the binder

for values of $C_3 \leq 1.0$, C_3 may be approximated by the ratio described.

The C_4 factor relating asphaltene content has been directly taken from Verstraeten's work. The values are shown in Figure 24. The last factor, C_5 , is in essence the adjustment factor used to account for differences between constant stress laboratory test results and in situ pavement performance. As discussed previously under the "University of Nottingham" discussion, the magnitude of this variable is not very well defined.

One final but interesting result published by Kirk in 1967[35] dealt with the effect of temperature upon fatigue performance. It has been stated that Kirk's tests with <u>continuously applied sinusoidal</u> loading verified the "strain criterion" of Pell (that is, a unique $\varepsilon - N_f$ curve, independent of mix stiffness upon constant stress results). However, a specially built fatigue test unit was developed to apply impulse loads (repeated applications) to fatigue specimens. Kirk reported that when pulse loads were applied, different strain-life relationships were clearly established for each temperature and thus the results did not verify the strain criterion.[35] Interestingly enough, no further results regarding this were found by the author of this report. Kirk's results are shown in Figure 30.

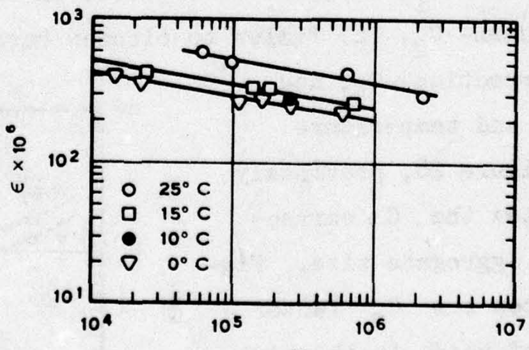


Figure 30. Strain plotted against number of loading cycles to failure. Impulse loading

THE ASPHALT INSTITUTE

Background. The Asphalt Institute (TAI) has been engaged in

research dealing with fatigue of bituminous mixtures for approximately 10 yr. In general, the methods of analysis have had as a salient objective the development and verification of a structural design fatigue subsystem. This is in contrast to any intensive study dealing with fundamental effects of mix properties upon fatigue performance. Two basic modes of fatigue research have been studied by The Asphalt Institute. They are: (a) laboratory control stress and strain fatigue testing on various mixtures, and (b) the development of a fatigue criteria based upon an extensive multilayered elastic analysis of the American Association of State Highway and Transportation Officials (AASHTO) Road Test. These criteria were developed by R. I. Kingham[39] and slightly modified by M. W. Witczak[40] for inclusion in the recent TAI manual publication, MS-11, "Full-Depth Asphalt Pavements for Air Carrier Airports."[41]

The laboratory fatigue testing apparatus is basically identical to that developed by Deacon and used in the University of California (Berkeley) fatigue research. The apparatus used for both controlled stress and strain is a repeated flexural test with third-point loading. A modification to the equipment allowed the use of larger rectangular beam specimens than used in California. The specimens were 3 in. by 3 in. by 15 in. (in comparison with the 1-1/2-in. by 1-1/2-in. cross section at Berkeley). Recent research by Kallas and Puzinauskas[42] has shown that strain-life regression equations using the larger specimens have comparable or better statistical correlations with much fewer specimens (6 to 9 per test) in contrast with the smaller beam specimens (19 to 53 specimens per test).

The type of load applied is a pulse variety (haversine) with typical load-dwell time of 0.1 sec (load) and 0.4 sec (dwell rest time). This is equivalent to 2 applications per second. As dynamic deflections are continuously monitored, flexural stiffnesses are calculated on each specimen for any number of stress applications. Approximately 20 different mixes have been tested in fatigue behavior. Additionally most of the later testing has been conducted at three temperature levels. The normal range of temperature has been from about 40°F to 85°F.

Results--Laboratory Testing. It has been noted that a major systematic experiment was not conducted by TAI to investigate the effect of mix properties upon fatigue behavior. Rather, lab fatigue tests have been confined primarily to specimens obtained from actual field pavements or molded in the laboratory to simulate field pavements. Most of these pavements have been investigated in a more thorough and complete overall performance model. Because of this objective, few conclusions, if any, can be directly drawn from the work regarding mix properties.

However, one important fact determined from fatigue work at multitemperature levels is that unique or different strain-life relationships occur for each test temperature. This, of course, is in contradiction to the so-called "strain criterion" hypothesis advocated by Pell and reportedly verified by Kirk and Verstraeten. The results by the Institute are likewise supported by results of Monismith and Kirk's limited study with impulse load techniques. Figures 31 and 32 illustrate typical temperature-dependent fatigue results for different mixes. (It can be seen that, in general, the results for the 80°F temperature depart significantly from the results at 40°F and 60°F.) In addition, Figure 32 illustrates that a temperature effect appears to occur not only

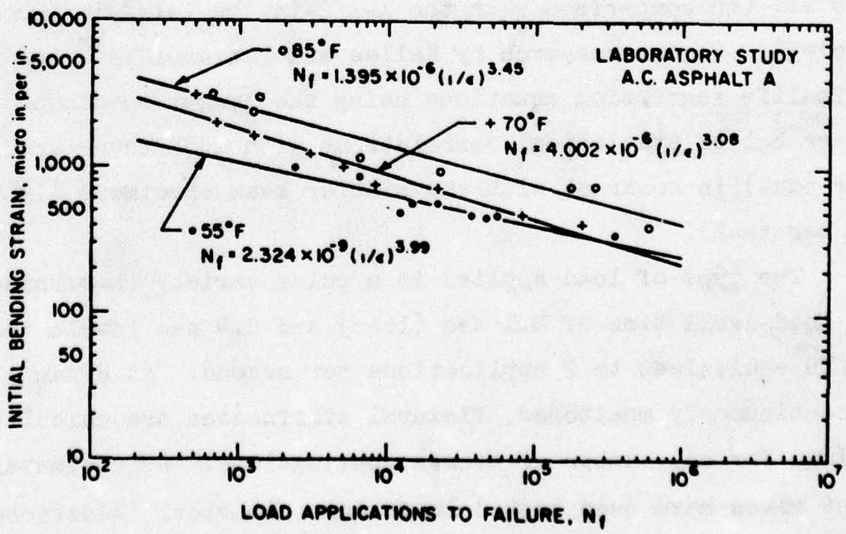


Figure 31. Strain-fracture life fatigue results for various temperatures for laboratory study asphalt mix (from Kallas and Puzinauskas[42])

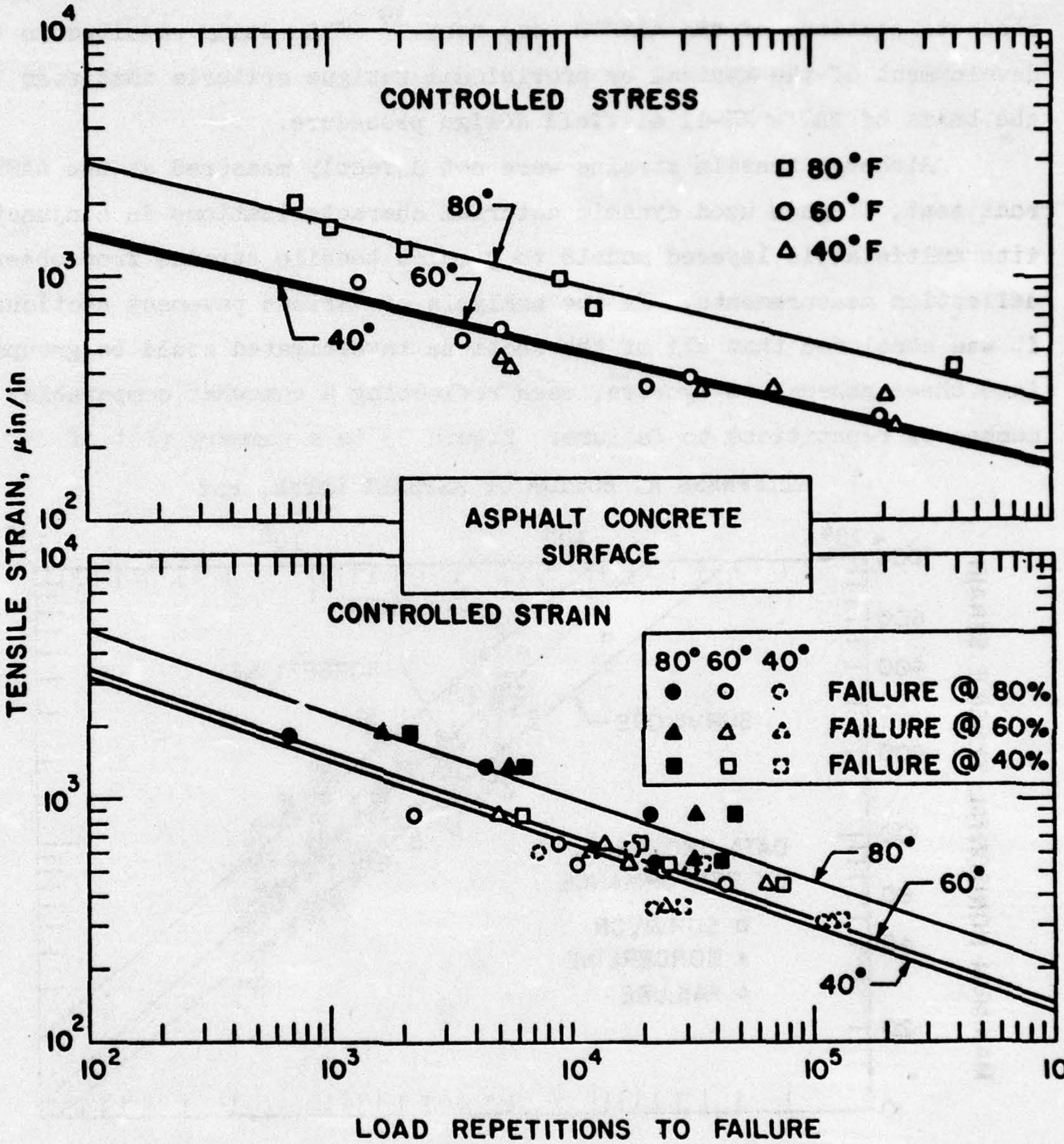


Figure 32. Results of flexural fatigue tests--asphalt concrete surface (from Kingham[39])

for controlled stress but for controlled strain tests as well.

Results--Typical Fatigue Criteria. In the early 70's it was recognized that although much research was being conducted internationally on laboratory fatigue testing, a big gap existed in directly applying these results to a structural fatigue design model. Accordingly, Kingham conducted an intensive study of the performance data on thick asphalt

concrete sections of the AASHTO road test.[39] This study resulted in the development of the typical or provisional fatigue criteria that form the basis of TAI's MS-11 airfield design procedure.

Although tensile strains were not directly measured at the AASHTO road test, Kingham used dynamic material characterizations in conjunction with multielastic layered models to predict tensile strains from observed deflection measurements. In the analysis of various pavement sections, it was concluded that all of the sections investigated could be grouped into three general categories, each reflecting a somewhat comparable number of repetitions to failure. Figure 33 is a summary plot of

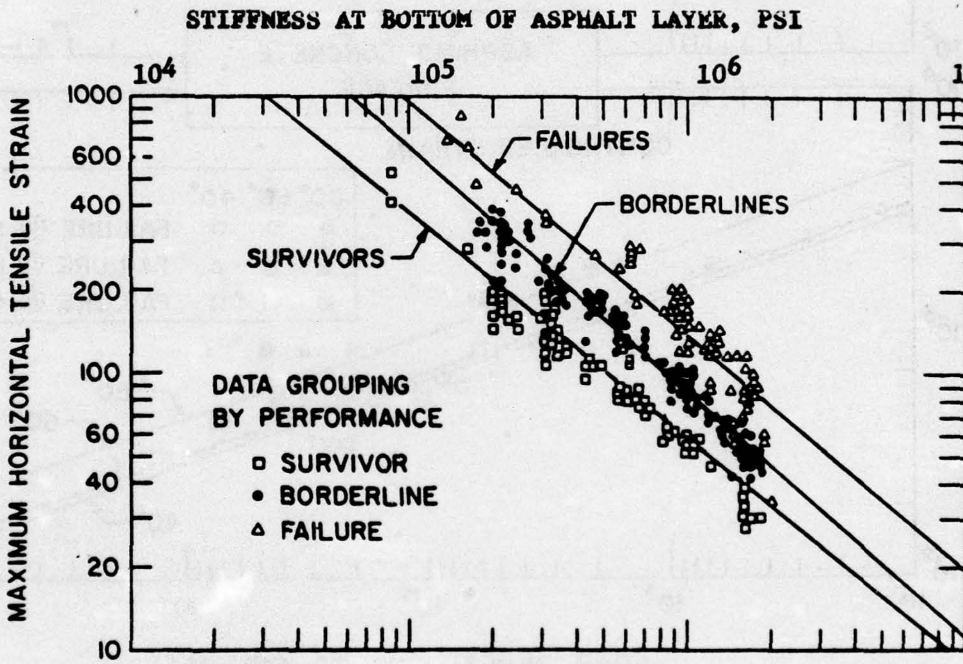


Figure 33. Strain-stiffness relationship for various pavement performance categories (from Kingham[39])

Kingham's results showing the predicted strain levels as a function of the asphalt stiffness for each performance classification. By assigning general N_f values to each category, a general relationship between tensile strain ε and failure life N_f and asphaltic stiffness E_1 was determined. This relationship was:

$$\log \varepsilon = 1.2458 - 0.67296 \log E - 0.0065461 \log N_f$$
$$- 0.034001 \log E \log N_f$$

Because failure repetitions were assigned to each performance category upon the basis of a terminal serviceability level of $p = 2.5$, it must be clearly understood by the reader that the results define a strain-life relationship denoting complete structural or, more precisely, functional failure of pavement systems. Thus they do not and cannot be compared with laboratory fatigue results that reflect only crack initiation.

It can be noted also that the analytical relationship developed by Kingham possesses an interaction product of stiffness and failure repetitions. This implies that sets of strain-repetitions curves for various stiffnesses would not result in parallel-type curves. In order to simplify the analytical treatment presented in MS-11, Witczak slightly modified the original Kingham expression.[40]

The design relationship adopted by TAI is:

$$N_f = ab^{q^d} \left(\frac{1}{\varepsilon}\right)^c$$

where N_f is the repetitions to failure, ε is the tensile strain, q is the mix (pavement) temperature, and a, b, c, and d are regression constants with values of 1.86351×10^{-17}, 1.01996, 4.995 and 1.45, respectively. As noted by the form of the equation presented, fatigue lines for various temperatures are parallel having a common slope (power) of $c = 4.995$. Of interest is the correlation in this value with that proposed by Verstraeten on laboratory fatigue tests of $0.217^{-1} = 4.61$. Figure 34 is a plot of the modified fatigue criterion (typical) defined by the previous equation. Because an analytical relation between dynamic modulus of a "typical" asphalt concrete mix and temperature was developed, the criterion could also be expressed in terms of stiffness rather than temperature.[40] Such a criterion is shown in Figure 35.

Detailed analytical studies conducted by Witczak have led to the development of the "effective temperature concept" for use in fatigue

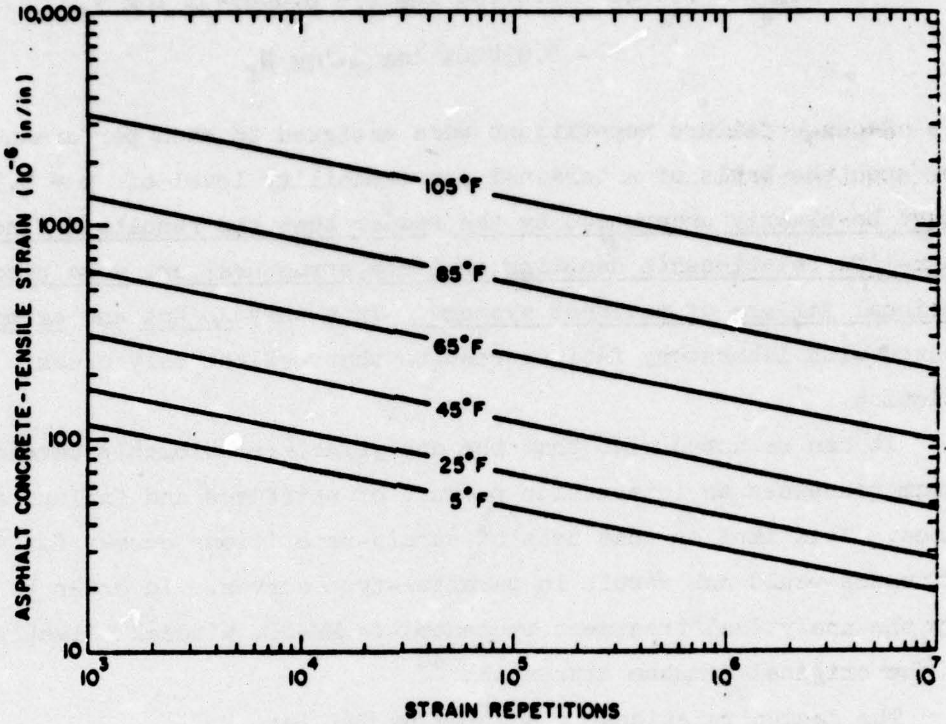


Figure 34. TAI typical fatigue criteria (from Witczak[40])

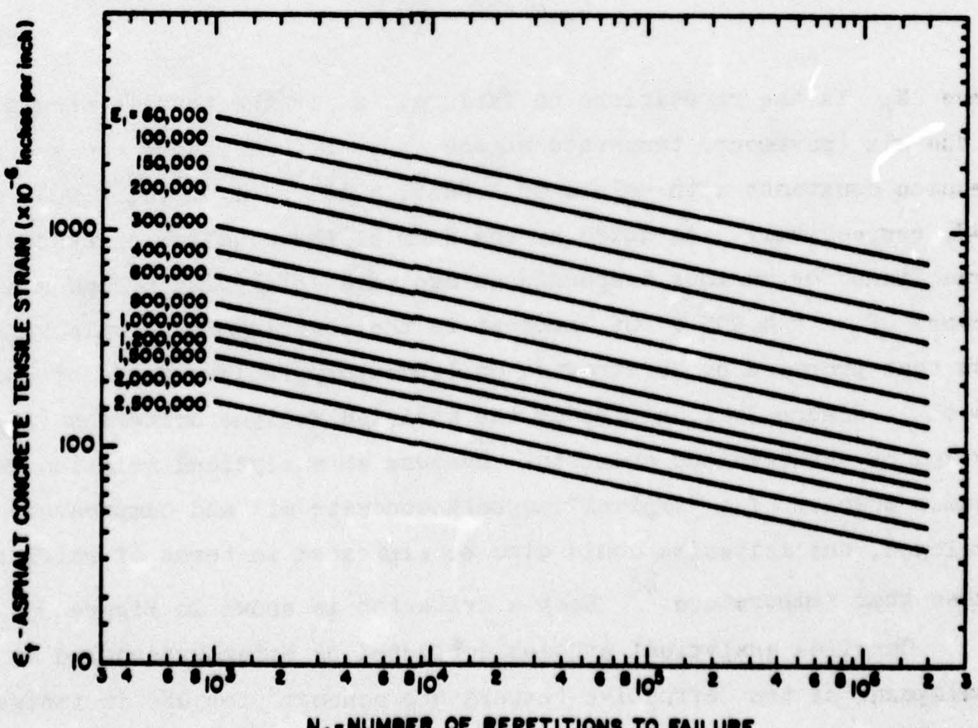


Figure 35. TAI typical fatigue curves (from Witczak[40])

analysis.[40,43] In essence an effective temperature for design q_e is defined as the unique pavement temperature at which the allowable number of repetitions to failure, obtained by using cumulative damage law with the multitemperature fatigue curves shown in Figure 34, is the same as the level of failure repetitions obtained by evaluating the pavement system at the same temperature (stiffness) for tensile strain and using the associated fatigue-temperature-life curve shown in the figure. Further work has shown that the q_e value is very close to the magnitude of the mean annual air temperature for any site.

Thus if a mean annual air temperature for a given location was 55°F, then the unique fatigue curve applicable to this location would be defined by the previous $N_f - \varepsilon$ equation for a $q \cong 55°F$. Additionally, it is imperative that in using multilayered theory to predict the tensile strain for the load-pavement system, a value of E_1 (asphalt concrete modulus) be used that corresponds to the effective temperature ($q_e = 55°F$). For conditions of the procedure developed by TAI, the E_1 may be found from:

$$E_1 = \frac{3.8 \times 10^6}{1.0046^{q_e^{1.45}}}$$

Design Applications and Verifications. The most immediate design application of the fatigue research conducted by TAI has been published in their MS-11 airfield design procedure.[41] The reader is referred to this publication for further details on the use of the modified Kingham criteria. Fatigue verfication studies have been conducted, however, using the results of both direct laboratory fatigue results as well as the typical criteria. These verification studies have included the analysis of nine different pavement section-performance results at the Washington State University (WSU) Test Track[44] as well as a thorough study conducted at Baltimore-Washington International Airport (formerly Friendship).[31,45]

The comparison study at WSU involved a performance analysis of three sections each of three different base types (sand asphalt, asphalt

Table 2

Predicted and Observed Performance

Base Type	Section	Surface and Base Thickness in.	Failure Criteria--Repetitions to Failure: Lab Controlled Stress	Lab Controlled Strain	Field AASHTO Moduli	Field WSU Moduli	Actual[5]
Sand asphalt	1	3-2	132,100	(162,784+)	3,400	151,700	144,660
	2	3-4	(190,801+)	(190,801+)	9,100	(190,801+)	159,789
	3	3-6	(220,189+)	(220,189+)	54,000	(220,189+)	175,620
Asphalt concrete	5	3.1-0	153,300	157,100	3,400	14,800	47,391
	6	3.1-2	164,800	(158,137+)	14,800	103,000	148,887
	7	3.3-3.5	174,900	(170,710+)	82,500	165,800	161,262
Crushed stone	9	3-4.5	147,000	152,700	390	3,400	12,000
	10	3-7.0	150,300	156,700	390	3,400	47,391
	11	3-9.5	153,700	158,000	390	3,400	48,000
	12	3-12	156,700	160,100	390	3,400	49,104

Note: (+) Further analysis periods are required for the failure prediction since cycle ratios were less than 0.85 at the end of computation.

concrete, and crushed stone). Table 2 is a summary of the predicted and observed repetitions to failure. It should be noted that fatigue cracking was verified to have actually occurred and the actual repetitions identified in the table are those associated with repetitions to initial surface cracking.

In the table, the first two columns under the "Failure Criteria" heading are predictions based upon laboratory controlled stress and strain fatigue curves. The last two columns refer to predictions made with the Kingham criteria. Because the original criteria were developed with E_1 as the primary variable (see Figure 33) and fatigue data were based upon multitemperature conditions, relationships between E_1 and q were necessary. The column identified by "Field AASHTO Moduli" refers to the typical E_1-q relationship previously noted. The "Field WSU Moduli" column refers to the specific E_1-q relationship obtained directly on the WSU pavement materials.

From this table, it can be seen that even lab-controlled tests tended to overpredict (be unsafe) in regards to observed repetitions to failure. However, it should be pointed out that the exact subgrade modulus that existed in situ during the test was felt to be quite difficult to predict. Thus some of the error may quite possibly be due directly to this factor. Also it can be seen that, from a practical viewpoint, laboratory predictions using lab fatigue data were in much closer agreement with observed values for the two full-depth sections (particularly for the asphalt concrete base). The worst prediction, using laboratory results, was thus seen to occur for the crushed stone bases (conventional flexible pavement types).

In comparing the use of the "typical fatigue criteria," the following can be observed. For most of the nine sections, conservative fatigue predictions were found to exist. In addition, when the actual material E_1-q relationship was combined with the Kingham criteria a much closer agreement was obtained.

In general, the results shown are viewed as somewhat encouraging, particularly for the use of the typical fatigue criteria proposed. Results from lab fatigue testing were not quite as good, especially for

the crushed stone bases. Because it has been previously noted that the selection of a proper subgrade modulus may have been responsible for this discrepancy, Kingham also investigated lab predictions (controlled stress) using possible maximum and minimum subgrade moduli. Table 3 summarizes this study and definitely indicates that the possibility of a much better prediction from lab tests could occur due to a use of a somewhat lower subgrade modulus.

The verification at Baltimore-Washington International Airport used both laboratory fatigue tests and the typical fatigue criteria shown in Figure 35. The results of the use of laboratory fatigue results (constant stress) have already been discussed under the University of California section of this report. This analysis resulted in a very favorable prediction (6,000 to 7,000 repetitions from lab criteria) relative to 9,000 actual repetitions to initial cracking and 13,500 repetitions to "failure."[45] Use of the typical TAI criterion resulted in an even closer correlation.[31] For this study, a predicted failure repetition level of 9,400 repetitions was obtained. This value again should be compared with the 9,000 and 13,500 repetitions observed for "initial cracking" and "failure."

In summary, it can be conclusively stated that a generally good (conservative) estimate of fatigue cracking is obtained by the use of the typical fatigue criterion developed by TAI. The use of laboratory-controlled stress fatigue curves, although not quite as good as the typical criterion, certainly appears to yield results that are somewhat comparable to observed performance particularly for full-depth asphaltic pavements.

SHELL OIL COMPANY

Background. One of the most intensive research efforts dealing with the use of asphalt in pavement systems has been conducted by the Shell Oil Company. As referred to in this report the Shell Oil Company consists of affiliate, research and developmental groups in the United States, Canada, England, France, and the Netherlands. By far, the major research reports have been under the direct auspices of the

Table 3

Predicted Performance Comparison

Using Lower Subgrade Modulus

Laboratory, Controlled Stress Failure Criteria

Base Type	Section	Surface and Base Thickness in.	High E_s * Damage Ratio	High E_s * Repetitions	Low E_s ** Damage Ratio	Low E_s ** Repetitions	Observed†
Sand asphalt	1	3-2	1.0	132,100	1.0	25,200	144,660
	2	3-4	0.85	190,800	1.0	90,700	159,789
	3	3-6	0.34	220,200	0.52	153,700	175,620
Asphalt concrete	5	3.1-0	1.0	153,000	1.0	67,800	47,391
	6	3.1-2	1.0	164,800	1.0	158,000	148,887
	7	3.3-3.5	1.0	174,900	1.0	166,800	161,262
Untreated	9	3-4.5	1.0	147,000	1.0	18,700	12,000
	10	3-7.0	1.0	150,300	1.0	22,200	47,391
	11	3-9.5	1.0	153,700	1.0	25,200	48,000
	12	3-12.0	1.0	156,700	1.0	25,200	49,104

* High subgrade modulus E_s : asphalt bases 15,700 psi, crushed stone base 7,700 psi.
** Low subgrade modulus E_s: asphalt bases 6,800 psi, crushed stone base 3,800 psi.
† Observed number of load repetitions to first cracking.

Shell Koninklijke Laboratory in Amsterdam.

Within the past quarter century, close to thirty individuals have published relevant research dealing with asphalt and asphaltic mixtures. Based upon an overall literature review, it appears that research[38,46-74] has been equally focused upon all facets of pavement design and performance, to include:

a. Fundamental rheologic behavior.

b. Dynamic testing and materials characterization.

c. Deformability and fracture theories.

d. Use of multilayered elastic theory.

e. Overall structural design subsystems.

The foundation of almost all research findings reported by Shell is based upon the stiffness concept of bitumens developed by Van der Poel in 1954.[38] In contrast to direct research on asphaltic mixtures, Shell researchers have concentrated much of their effort directly on the asphalt cement (bitumen). Huekelom has stated his belief,

> ...that the more complicated behaviour of asphalt mixes - including effects of bitumen content, grading and type of minerals, degree of compaction etc...can be understood only when the properties of the asphalt cement are fully known.

Thus, relative to fatigue research, several studies have focused directly upon the fatigue behavior of bitumen.[38,48,59,69,71] Ironically, it also appears that most of the original reported fatigue work on asphalt and asphalt mixtures is basically related to the development of the original "provisional type" of fatigue criteria introduced by Nijboer in 1960. It is these typical fatigue criteria that have been used to develop the limiting fatigue strain-life criterion concept. This latter innovation formed the basis for establishing a structural design system for highway and airfield pavements[54,66,67] that uses both fatigue and deformation criteria for design.

However, recent laboratory fatigue work appears to have been directed to laboratory test methods and their effects upon mix variables, derivation of an energy-related fatigue criteria, study of the effect of rest periods upon fatigue life, and a comparison of laboratory fatigue

behavior from conventional tests with results obtained on specially constructed devices to monitor fatigue behavior more precisely as it occurs in situ. This has been accomplished with a circular test track and a device termed the Wheel Tracking Test (WTT).

While discussions in this report concerning other investigators have attempted to detail fatigue and stiffness test apparatus and methods of calculation, it is nearly impossible to do so with the Shell organization. This is primarily due to the fact that numerous methods and procedures have been used in their investigations throughout the years. The following sections summarize the major results dealing with (a) the development of the provisional fatigue curves and limiting strain criteria and (b) results of recent laboratory fatigue testing.

Results--Typical Fatigue Criterion. Some of the earliest reported fatigue work by Shell was done by Nijboer and Saal and Pell in 1960. Thus, fatigue research by Shell is over 15 yr in progress. Based upon the results of Nijboer, Huekelom and Klomp published a provisional fatigue-life relationship as a function of mix stiffness in 1962.[50] Only a very limited amount of information is available to indicate under what precise test conditions and method of moduli computations the results were obtained. In 1964, Huekelom and Klomp[55] alluded that the results were obtained from fatigue measurements with a 3-point electromagnetical vibration system. The system used a continuously applied sinusoidal bending load on what appears to be extremely small bar specimens. The specific method used to compute initial strains was not found in the literature. Thus, the author is uncertain as to whether the stiffness nomograph was used, or whether direct dynamic stiffnesses were measured on the same or separate fatigue specimens.

Figure 36 illustrates provisional fatigue criteria introduced by Huekelom and Klomp from Nijboer's work in 1962. As noted in the diagram, the results are considered applicable for a void content of 5 percent. Figure 37 shows these same criteria expressed in the more familiar log e/log N relationship.

Results--Limiting Strain Criterion. During the approximate same time period as the development of the provisional fatigue curves, Shell

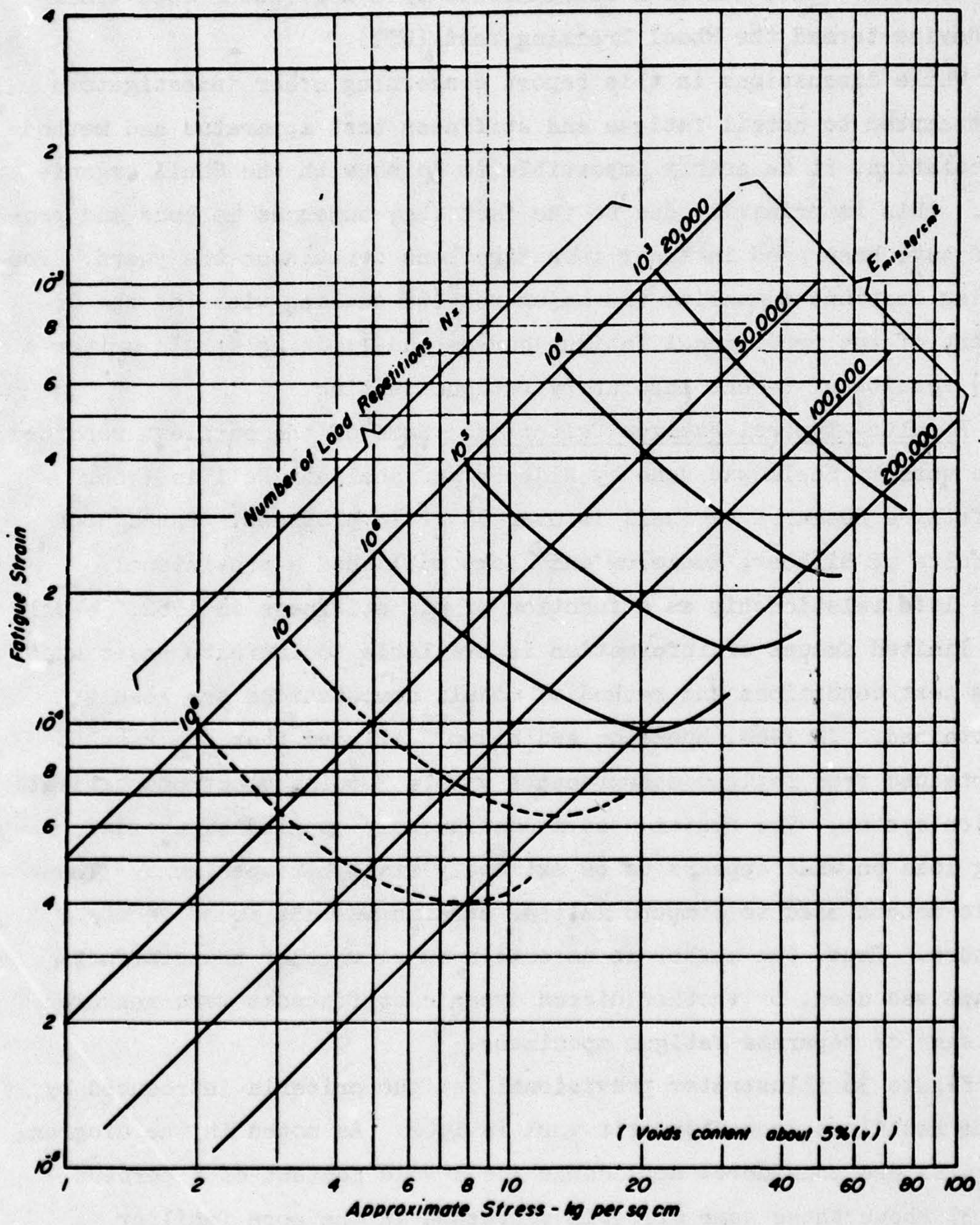


Figure 36. Provisional fatigue relationship for bituminous base materials (after Huekelom and Klomp[55])

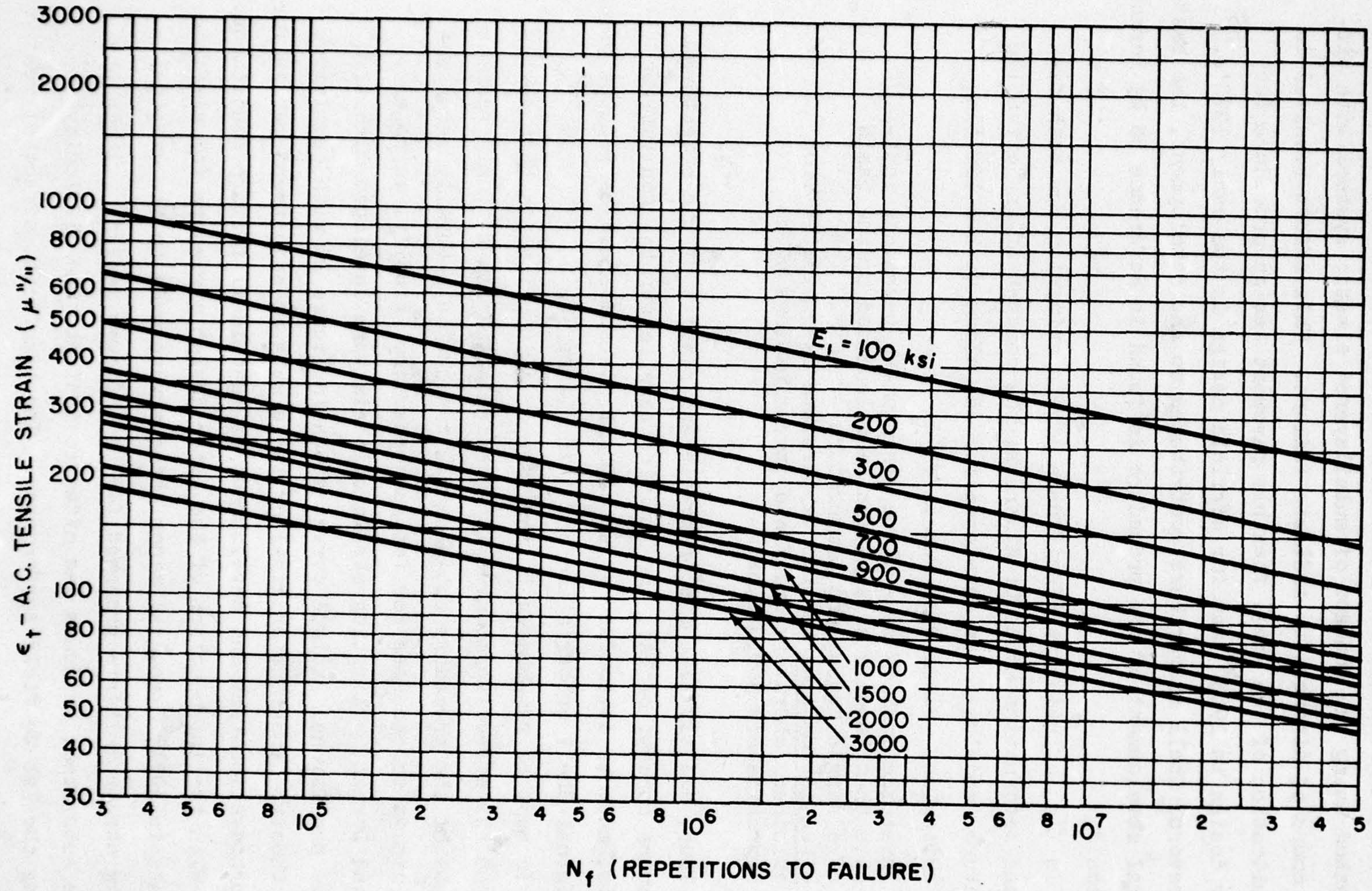


Figure 37. Shell Oil Company, asphalt concrete fatigue relationship (after Huekelom and Klomp[55])

researchers were engaged in the development of a structural pavement design method using the concept of multilayered elastic systems with differing modes of distress to indicate failure. This effort resulted in the publication of the Shell flexible pavement design procedure for highway design in 1963[54] and for airfield design in the early 1970's.[66] While several Shell researchers contributed to the development, the basic technical development of the procedure is found in Reference 58 by Dormon and Metcalf.

In lieu of using multistiffness strain criterion for both fatigue and deformation modes of distress, the concept of an "effective temperature (modulus)" was developed for use along with a limiting strain criterion. Based upon Metcalf,[56]

> Investigations of several representative pavements indicate that tensile strains can be calculated on the basis of a single equivalent temperature of 50°F. The <u>effective modulus</u> at this temperature is estimated from combined laboratory and field investigations to be approximately 900,000 psi.

Thus, exactly like the principle and results of the effective temperature concept of TAI, the limiting tensile strain, for fatigue analysis, must be determined at an asphalt mix stiffness of 900,000 psi. The original Shell criteria are shown in Figure 38.

It must be remembered by the user of this criterion that deviation from the effective modulus in a design situation not only results in misuse of the procedure but also will result in generally unconservative design thicknesses for fatigue behavior. Finally, as noted by Klomp and Dormon,[57] limiting strain criteria were developed directly from the provisional fatigue curves shown in Figure 36 with cumulative damage theory. Thus the original criteria make no adjustment per se for crack propagation time and development of functional failure conditions.

With the introduction of the later design procedure for airfield pavements in 1971,[66] several minor changes were made to the original limiting strain criteria proposed for the highway design. Within the fatigue subsystem, evidence was cited to indicate the difficulty in relating the lab to field performance. The beneficial effect of rest

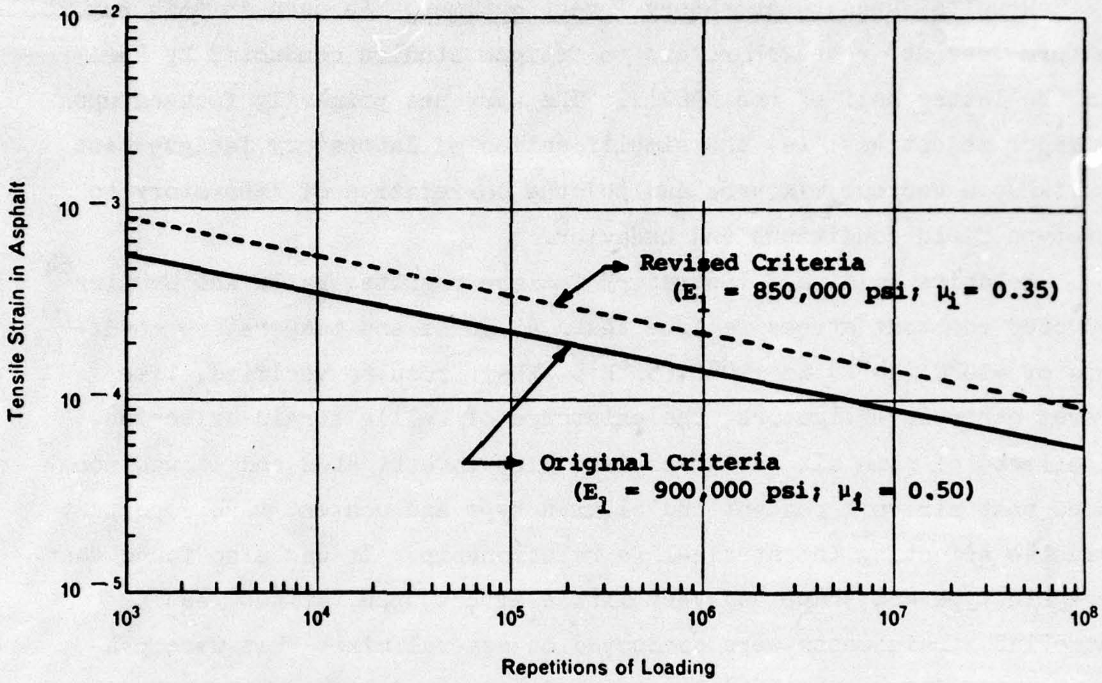


Figure 38. Shell Oil Company limiting fatigue strain criteria

periods as well as the possible healing effect of fatigued asphaltic mixtures were also cited. In addition, minor changes were made to reflect a new effective modulus of 60,000 kg/cm^2 (≈850,000 psi) as well as reducing the Poisson's ratio (ν_1) value of all layers from 0.50 (original criteria) to 0.35 (revised criteria). Because the Airfield Design Manual illustrates structural pavement designs for various aircraft at a single repetition failure level of $N_f = 10^6$, the revised limiting asphalt tensile strain was increased, at 10^6 repetitions, from the original 150 μin./in. to 230 μin./in. This revision, while only noted for the one repetition level, amounts to about a one log cycle shift (i.e., factor of 10) for the revised criteria. Again, it must be understood that the exact conditions of the revised criteria, relative to E_1 and ν_1, be followed in any design situation. The revised criteria are likewise shown in Figure 38 assuming the one cycle shift in failure repetitions.

<u>Results--Recent Laboratory Investigations.</u> As used in this report, the term "recent" research refers to fatigue studies conducted by Shell from the latter half of the 1960's. The work has primarily focused upon two major objectives: (a) the simplification of laboratory fatigue test results upon various mixtures and (b) the correlation of laboratory to prototype field conditions and behavior.

Relative to direct laboratory fatigue results, Bazin and Saunier[64] conducted constant stress fatigue tests at 50 Hz and temperature conditions of -10°C (14°F) to +10°C (50°F). Their results verified, like several other investigators, the existence of Pell's strain criterion. The effects of some mix variables were also investigated and it was concluded that air void content and bitumen type and content were important variables affecting the strain-life relationship. It was also found that aggregate type and shape had very little effect upon fatigue results. Controlled strain tests were conducted on several mixes that were previously tested in controlled stress conditions. It was found that the controlled strain results were 2 to 3 times the fatigue life obtained from controlled stress tests. In the test method, trapezoidal and rectangular beam specimens were subjected to sinusoidal loading.

One of the more important contributions by Bazin and Saunier was their study dealing with the healing characteristics of both broken (fractured) and fatigued specimens. It was shown that specimens subjected to a nominal compressive stress, at a given temperature, did possess the capability to heal for both fractured and fatigued specimens. Figure 39 illustrates these results for the fatigue tests at 10°C (50°F). The obvious influence of rest time upon the healing properties is readily shown.

In 1972, Van Dijk et al.,[69-71] reported laboratory results demonstrating the rather marked increase in fatigue life of asphalt specimens simply due to the effect of rest periods between load pulses. With a programmable function generator, a stress wave pattern, shown in Figure 40a, was used to simulate as nearly as possible actual stress conditions in laboratory specimens. Figures 40b and 40c illustrate the magnitude of the increased fatigue life as a function of the ratio of rest time T_r to load pulse time T_w for two different mixes.

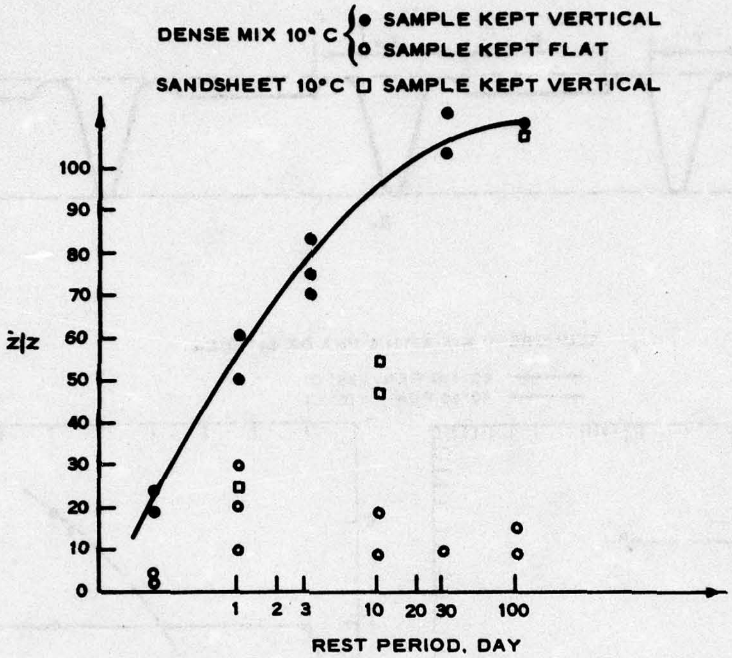


Figure 39. Effect of healing upon fatigue life (from Bazin and Saunier[64])

Van Dijk's latest publication, presented in 1975,[74] gave a summary of fatigue results based upon the concept of deformation energy rather than strain or stress. This energy concept was earlier suggested by Heukelom in 1966.[59] Previous work by Shell investigators on pure bitumen fatigue indicated that regardless of whether constant load or deflection tests were used for fatigue, the total dissipated shear energy per unit value was identical. Applying this concept to bituminous mixtures, the dissipated energy per unit volume per cycle of load may be expressed as:

$$W_i = \pi \sigma_i \varepsilon_i \sin \phi$$

As discrete intervals are defined, the total dissipated energy per unit volume W_{fat} within the interval is

$$W_{fat} = \sum_{i=1}^{n} N_i W_i$$

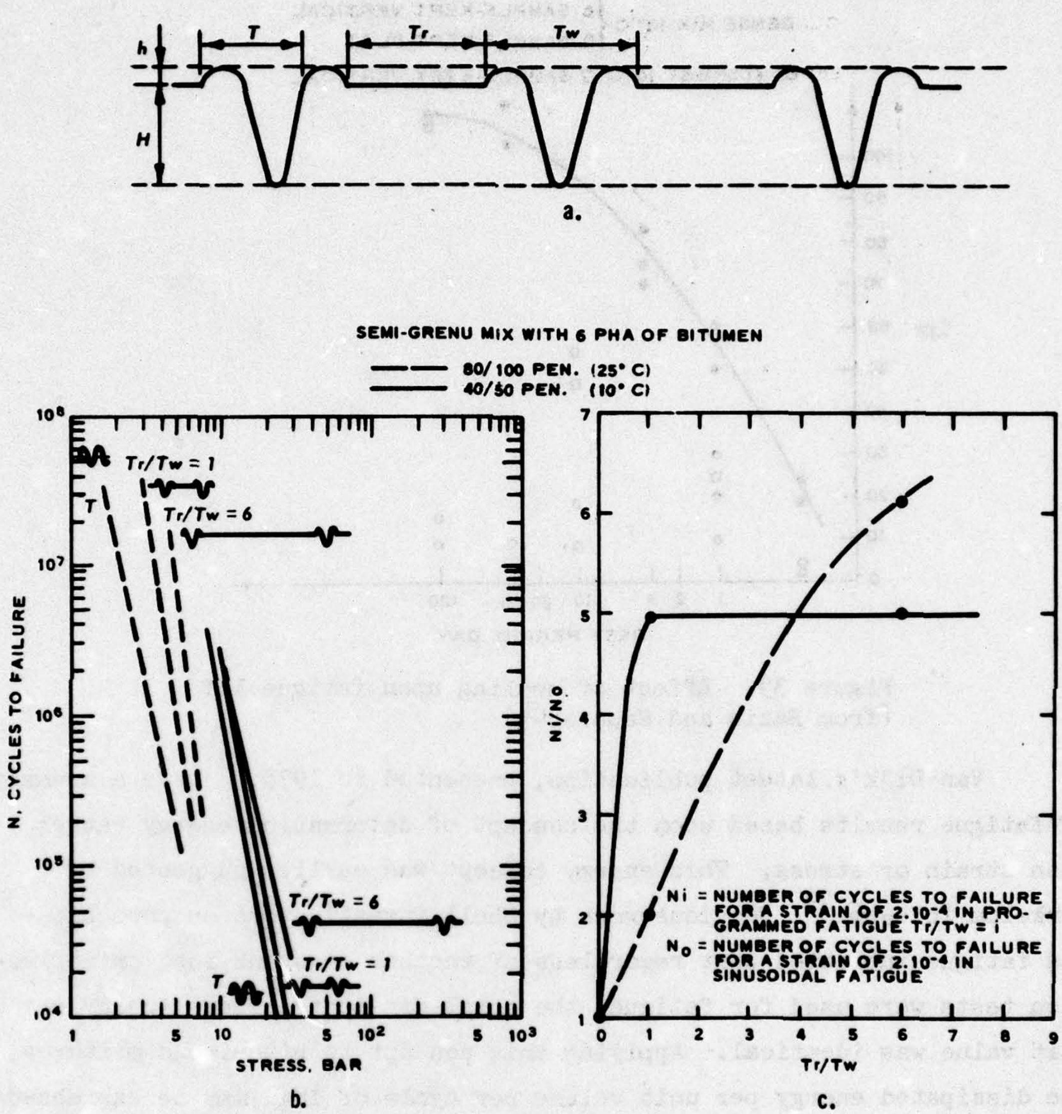


Figure 40. Effect of rest time upon fatigue results (from Van Dijk et al.[69-71])

where N_i is the number of applications within the interval. This principle was applied to all fatigue results conducted by Shell that allowed continuous stress σ_o, strain ε_o, and phase angle ϕ values to be obtained throughout the test. For a particular mix, it was found that a unique relationship between W_{fat} and N_{fat} (repetitions to failure) existed. Figure 41 illustrates such a result. Of importance is the fact

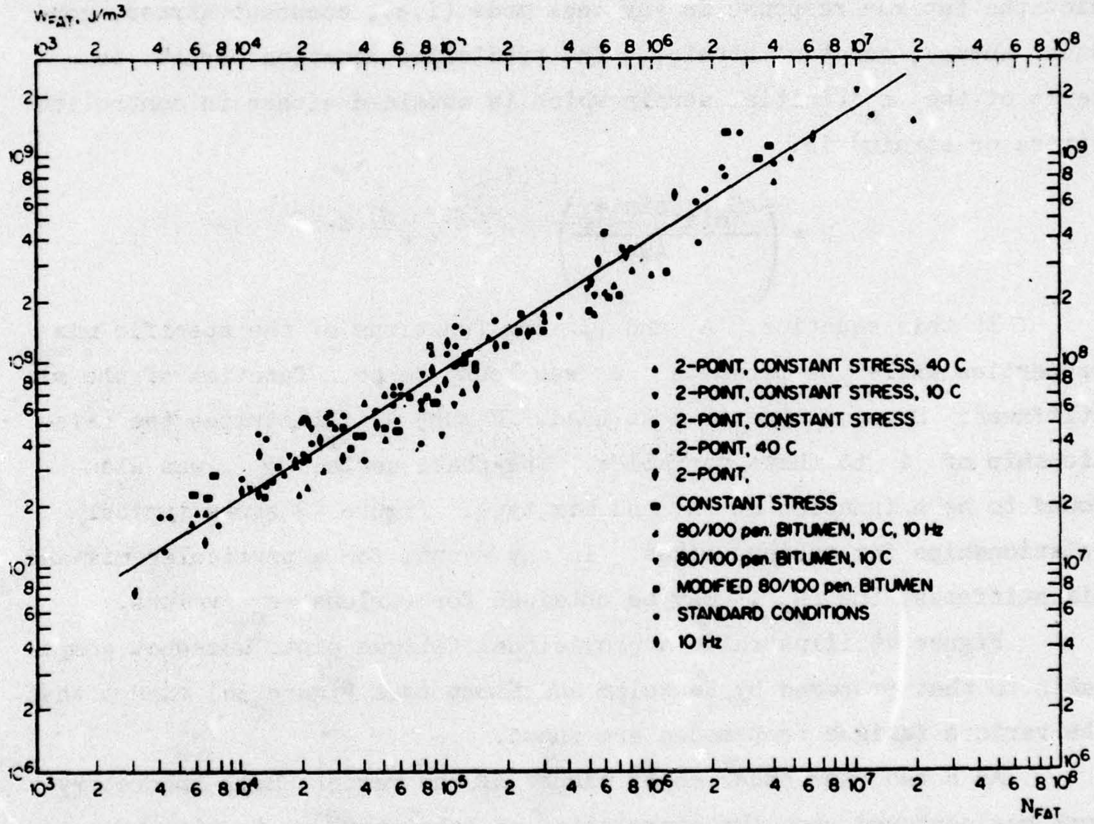


Figure 41. Deformation energy as a function of repetitions to failure for an asphaltic mix (from Van Dijk[74])

that the relationship shown is <u>independent</u> of test condition (2-point or 3-point bending), test temperature (10°C to 40°C), frequency (10 and 50 Hz), type of bitumen, and type of test (constant stress or constant strain).

As shown, a relationship of the form

$$W_{fat} = A \times N^z$$

exists for a particular mix. However, the constants A and z <u>are</u> dependent upon the mix properties similarly to the two constants (C or K and m) used to functionally relate strain to repetitions. For seven different mixtures evaluated, the range in A was 1.2×10^4 to 3.0×10^5 joules per meter^2 while z ranged from 0.51 to 0.68.[74]

Van Dijk proposed a procedure using these energy concepts to predict the fatigue response in any test mode (i.e., constant stress, constant energy, constant strain). The predictive equation for N in terms of the ε_o (initial strain which is obtained either in controlled stress or strain) is:

$$N = \left(\frac{\pi S_{mix} \sin \phi_o}{A\psi}\right)^{1/(Z-1)} \times \varepsilon_o^{2/(Z-1)}$$

In this equation, A and Z are functions of the specific mix properties while the parameter ψ was found to be a function of the mix stiffness, S , and type of test used. Figure 42 illustrates the relationship of ψ to these variables. The phase angle, ϕ_o , was also found to be a function of S and mix type. Figure 43 shows typical relationships for various mixes. In any event, for a particular mix and mix stiffness, the N_{fat} may be obtained for various ε_o values.

Figure 44 illustrates a provisional fatigue plot, somewhat comparable to that proposed by Heukelom and Klomp (see Figure 36) except that the various fatigue test modes are shown.

As noted, the other major thrust of the recent Shell laboratory work has centered upon the correlation of laboratory to in situ type behavior. Bazin and Saunier[64] used a circular test track in the laboratory along with constant stress bending tests to determine that a factor of 2 to 4 (in fatigue life) was obtained for the circular tracking test wheel load apparatus compared to laboratory bending tests.

Van Dijk[74] also investigated the use of a WTT to correlate laboratory to in situ type field behavior. Although the WTT is used in the laboratory, the applied wheel load obviously closely simulates the stress pattern (load rate as well as rest period) found in actual roadway conditions. In the study, strain gages combined with photographs (bottom

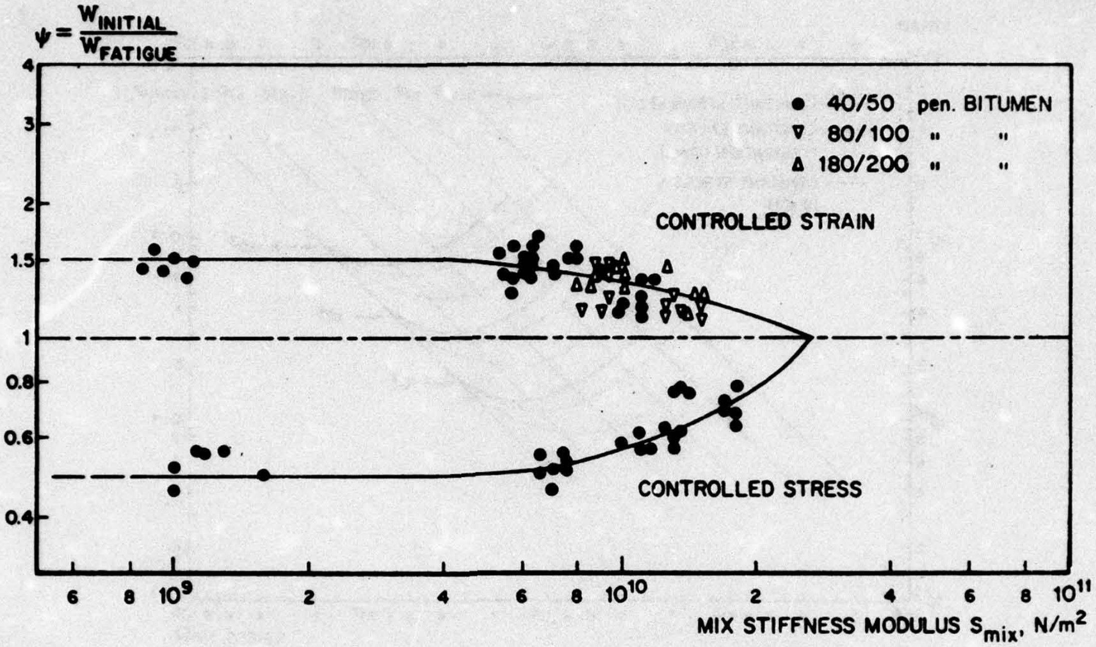


Figure 42. Relation of ψ and mix stiffness for an asphaltic concrete mix (from Van Dijk[74])

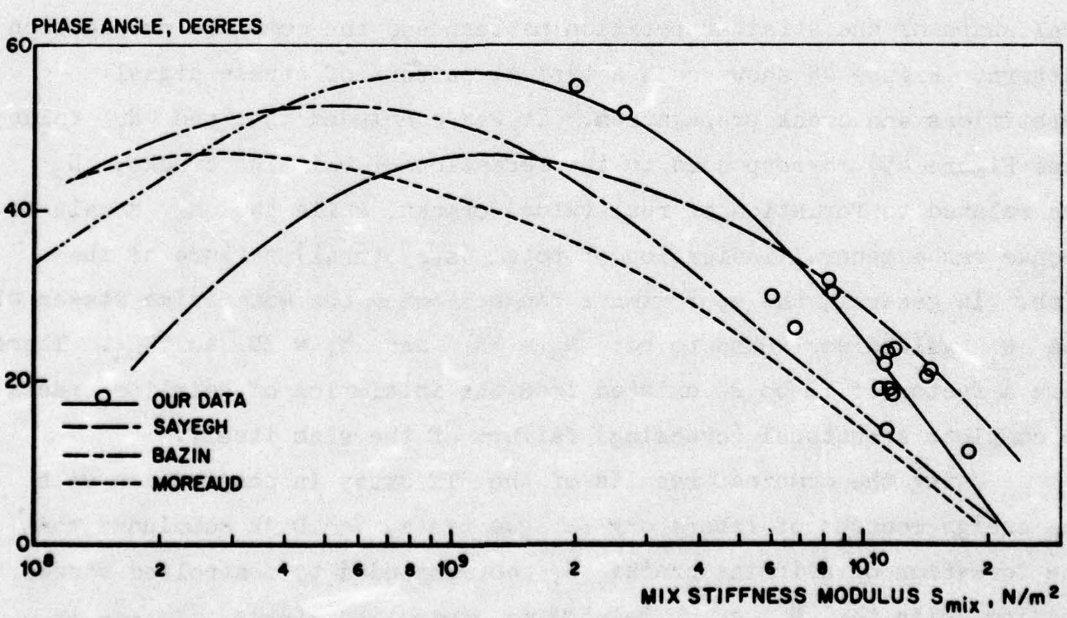


Figure 43. Phase angle between stress and strain versus mix stiffness modulus (from Van Dijk[74])

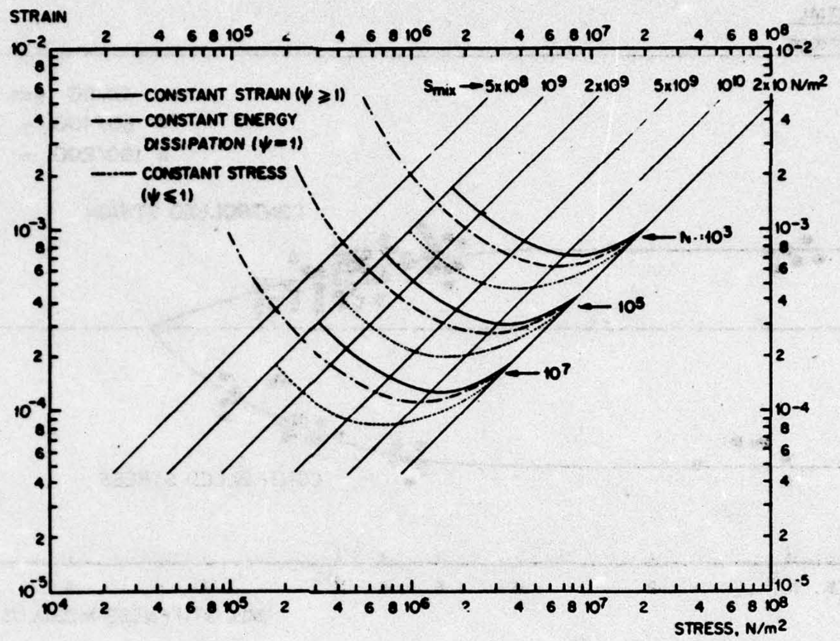


Figure 44. Chart to predict lifetime of an asphaltic concrete (from Van Dijk[74])

and top of slab) of crack patterns allowed a correlation between the general shape of the strain-repetition pattern and the complete propagation pattern. Figure 45 shows such a typical pattern of strain signal repetitions and crack propagation. It was concluded that the N_1 value (see Figure 45) corresponded to the formation of hairline cracks, N_2 was related to formation of real (wide) cracks, while the N_3 strain response was a general indication of total (structural) failure of the slab. In general, the approximate range between the successive stages of the N_1 values was found to be: $N_2 = 3N_1$ and $N_3 = 3N_2$ to $6N_2$. Therefore a factor of 10 to 20 existed from the initiation of hairline cracks to complete structural (cracking) failure of the slab itself.

Using the combined results of the WTT study in conjunction with the energy concept of laboratory fatigue tests, Van Dijk concluded that the formation of hairline cracks N_1 corresponded to controlled stress testing while the N_2 stage related to controlled strain. It was thus concluded that even for thick asphalt pavement sections (i.e., in the Controlled Stress Mode Factor), a more representative test for fatigue

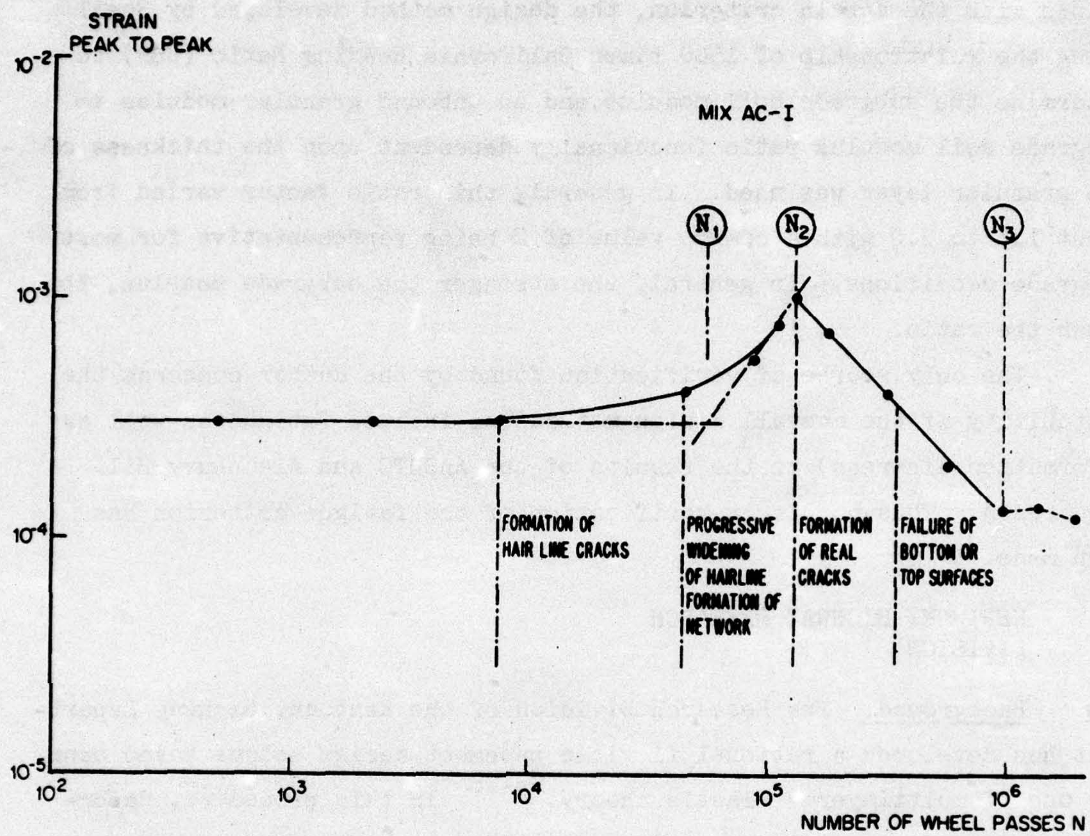


Figure 45. Crack development as a function of strain readings at equivalent number of wheel passes (from Van Dijk[74])

would be the <u>controlled strain</u> laboratory condition.

<u>Design Application and Verification.</u> The use of the limiting strain concept for analyzing pavement structure originated in the literature in the early 1960's with the introduction of the Shell design method (curves) for highways.[54] Since this time, the concept has appeared to have gained much approval from others as several other agencies have followed the ideas of the Shell design method.

Relative to fatigue analysis, the design curves presented for both flexible highway and airfield design[54,66] rely directly upon the limiting fatigue strain-effective asphalt concrete modulus concept. In addition to the mandatory use of the appropriate effective E_1 and μ_i

values with the strain criterion, the design method developed by Shell using the relationship of 1500 times California Bearing Ratio (CBR) to determine the subgrade soil modulus and an unbound granular modulus to subgrade soil modulus ratio functionally dependent upon the thickness of the granular layer was used. In general, this ratio factor varied from about 1.5 to 3.0 with a common value of 2 being representative for most subgrade conditions. In general, the stronger the subgrade modulus, the lower the ratio.

The only source of verification found by the author concerns the suitability of the overall design method (to include fatigue as well as deformation distress) to the results of the AASHTO and Alconbury Hill Road tests. Thus no direct verification of the fatigue criterion has been made.

KENTUCKY HIGHWAY RESEARCH DIVISION

Background. The Research Division of the Kentucky Highway Department has developed a rational flexible pavement design scheme based upon the use of multilayered elastic theory.[75-78] In this procedure, deformation and fatigue are accounted for in the design framework by the use of a limiting strain criteria. Both of these criteria have been developed from an analysis of the stress-strain response of pavement structures designed in accordance with the 1958 Kentucky design curves.[75] This design procedure (1958) is based upon the well developed, experience tested, thickness-equivalent wheel load-CBR procedure.

Results--Development of Fatigue Strain Criteria. The development of the fatigue criterion by Kentucky was based solely upon the interpretative analysis of other published fatigue works. Thus, there was no laboratory investigation of the fatigue phenomena conducted directly by the Research Division.

In the development it was noted that Van de Poel found a safe limiting strain of an asphalt mix to be approximately 100 μin./in. Additionally, the limiting strain criteria developed by Dormon and Metcalf[58] for 50°F at 10^6 repetitions was 145 μin./in. An equation

was developed for the original Shell limiting strain criteria at this critical temperature and it was found that an asphalt strain of 2240 μin./in. was computed for $N = 1$ (single catastrophic load fracture).

In addition, by plotting several computerized solutions of the multilayered problem for various pavement structures, it was found that for a given E_1 (asphalt modulus) the curves depicting structural influences appeared to all to converge at a single point near a strain of 2000 μin./in. Because of the similarity between this observed value and the strain from Dormon and Metcalf's equation for $N_f = 1$, an asphalt strain of 2240 was selected as the limiting strain for $N_f = 1$, independent of stiffness and structural influences. Figure 46 shows the results of this study.

Pursuing the development of the fatigue criterion, all theoretical results were related to one control pavement section having 33 percent asphaltic concrete, 23-in. total thickness, CBR = 7 and E_1 = 480,000 psi (to achieve matching predicted to observed Benkelman Beam deflections). For these conditions, a tensile strain of 140 μin./in. was obtained. From the 1958 Kentucky design curves, the controlled pavement section was associated with a repetition level of 8×10^6 equivalent 18-kip axle loads.

In Figure 46, a line drawn perpendicular to the modulus curves through the control pavement strain of 149 μin./in. represents an equal energy line and presumably locates limiting strains at 8×10^6 repetitions for various E_1 levels. Thus, having the strain value for $N_f = 1$ and the strains (for various E_1 values) at $N = 8 \times 10^6$, fatigue relationships were assumed to be linear between these two points on a log strain-log repetition curve. The Kentucky fatigue criterion developed is graphically shown in Figure 47.

Design Applications and Verification. The details of the design procedure recommended by the Kentucky Research Division may be found in References 76-78. Like all other limiting strain procedures, it is imperative that design input for any structure being analyzed follows the exact input used to develop the criteria. In the Kentucky procedure, it

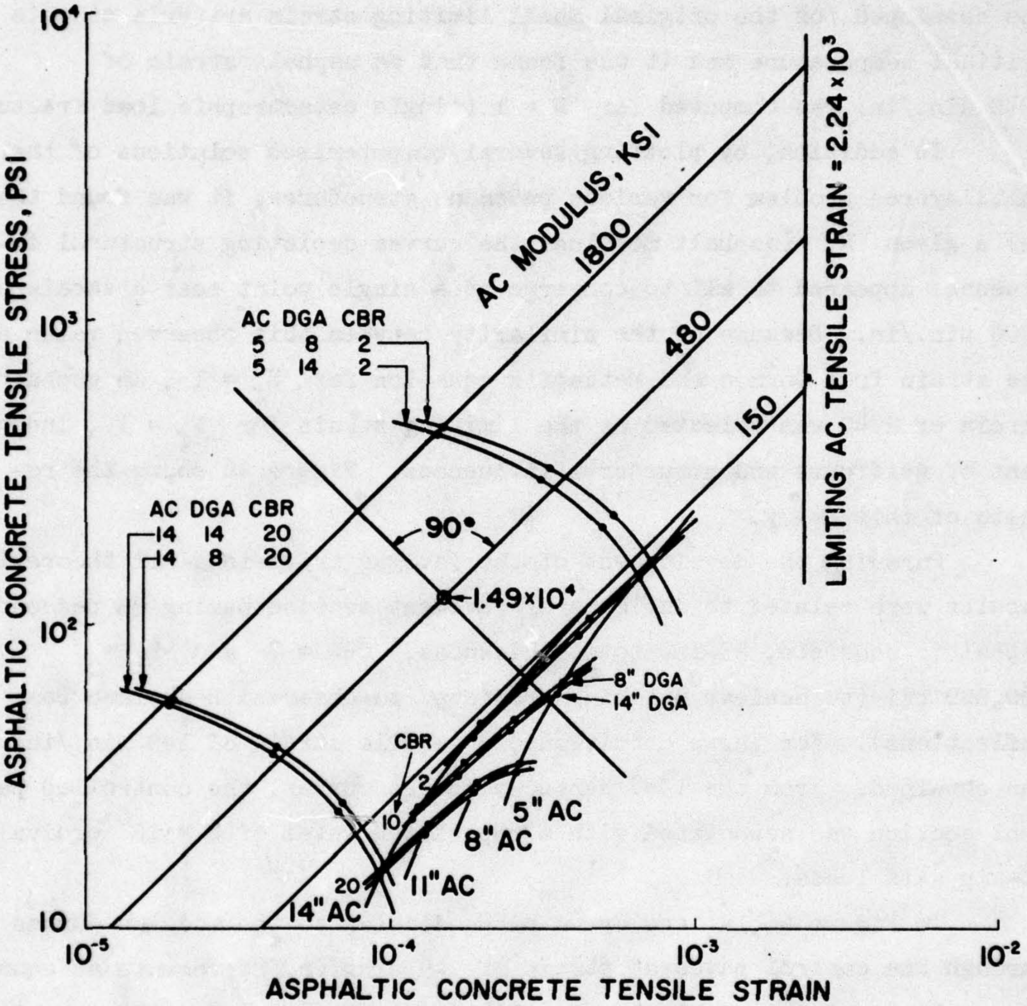


Figure 46. Asphalt tensile strains-stresses for various structures, CBR's, and asphaltic concrete moduli of elasticity (from Southgate, Deen and Havens[76])

is assumed that the Poisson's ratio of the asphaltic material and dense graded base material is 0.40, while for soil material the value was assumed to be 0.45.

Additionally, the subgrade modulus is correlated to CBR by the 1500 CBR equation. For unbound granular materials, the factor relating the base moduli to subgrade moduli is shown in Figure 48. Finally, when using the criteria for fatigue, it is imperative that the "effective" asphaltic concrete modulus used coincide with the following.

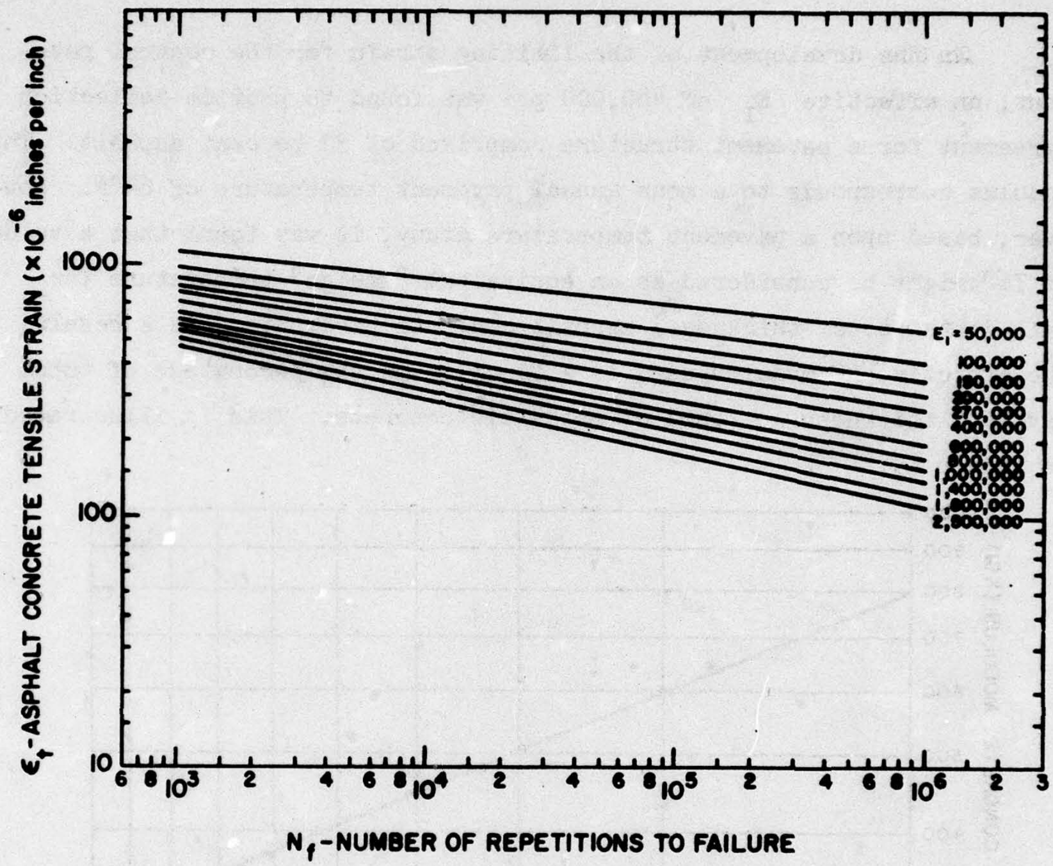


Figure 47. Typical fatigue criteria (Kentucky Highway Department[76])

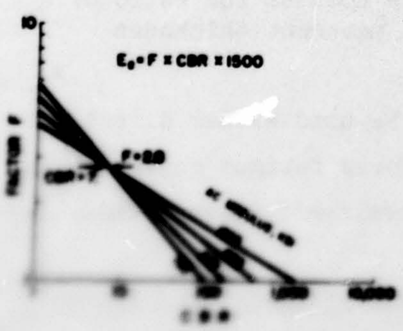


Figure 48. Relation of moduli of subgrade and moduli of granular base (from Havens et al.[78])

In the development of the limiting strain for the control pavement, an effective E_1 of 480,000 psi was found to provide deflection agreement for a pavement structure comprised of 33 percent asphalt. This modulus corresponds to a mean annual pavement temperature of 64°F. However, based upon a pavement temperature study, it was found that a value of 76° might be considered as an equivalent "design" temperature for full-depth (total thickness) asphalt concrete pavements. As a result, the effective AC modulus used is a function of the percentage of total pavement thickness composed of asphaltic concrete. This is illustrated in Figure 49.

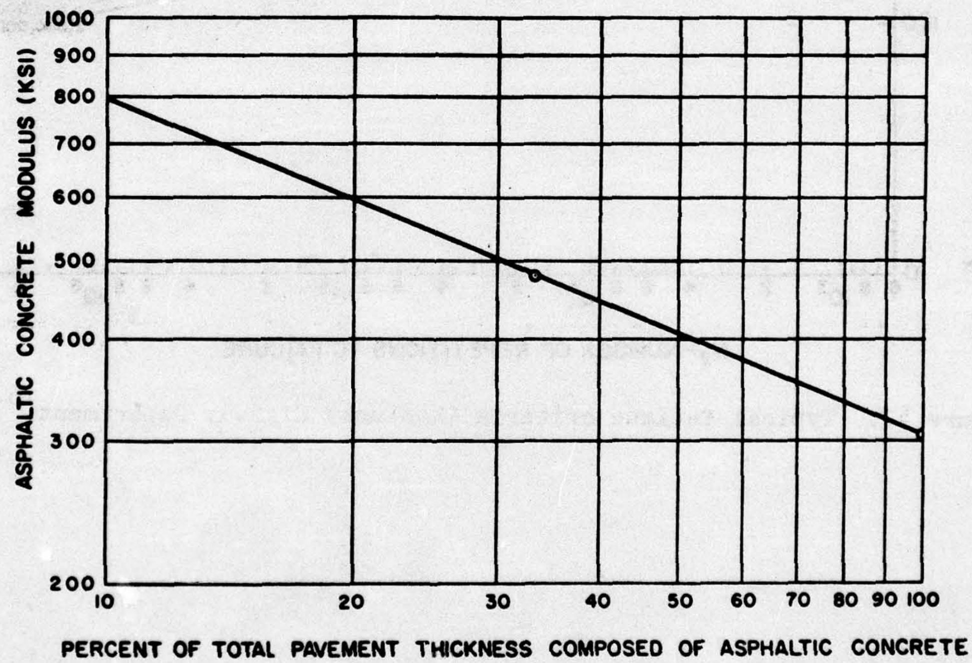


Figure 49. Weighting of asphalt concrete modulus for ratio of thickness of asphaltic concrete to total pavement thickness (after Kentucky Research Division[75])

As a result, the fatigue criteria can be used either directly in a cumulative damage model with the multistiffness fatigue curves shown in Figure 47 or with the use of a limiting strain-effective modulus concept previously discussed.

Because the criterion has been developed directly from previous experience with Kentucky's flexible pavement design curves, no other

verification type studies, to the author's knowledge, have been conducted by Kentucky. However, the author of this report has used the multistiffness family of fatigue curves in a cumulative damage study at Baltimore-Washington International Airport[31] to compare predicted and observed failure (fatigue) of an existing taxiway.

In this study, three different laboratory modulus-temperature relationships were used with the fatigue curves to predict damage. All three of these E_1-t relationships were found from laboratory tests conducted on cored samples of the taxiway. In general, there was a fairly good agreement between predicted and observed results. However, the results did depend upon whether dynamic modulus $|E^*|$, average flexural stiffness from bending tests E_s or a nonlinear flexural stiffness modulus $E_{s(\sigma)}$ was used.

When the nonlinear modulus-temperature model was used, a factor of safety (observed to predicted repetitions) of 1.12 was computed for initial surface cracking and an FS = 1.65 for failure (functional) repetitions. Use of the average flexural stiffness-temperature model resulted in FS = 0.65 and 0.96 for initial surface cracking and functional failure, respectively. Finally, the poorest prediction was obtained with the use of the dynamic modulus-temperature relationship. For this condition, FS = 0.24 and 0.35 were obtained from the two distress conditions.

OHIO STATE UNIVERSITY
B. S. Coffman

Background. A limited, but significant study, using laboratory fatigue testing and performance observations from prototype flexible pavement slabs subjective to repetitive loading has been conducted by Coffman.[79] Five asphaltic pavement sections (20 by 30 ft) were constructed and subjected to repetitive loads of one dynamic 10-Hz haversine pulse every second to simulate a continuous line of wheel loads traveling in identical wheel paths 50 ft apart at 40 mph. Observations were made visually on each slab as to the repetitions required for visible surface cracking. In addition, specimens were obtained from each

slab and tested in fatigue under the same loading wave forms at various temperatures.

Results. Figure 50 illustrates the general shapes of the fatigue apparatus and specimens used. As shown, the specimens are trapezoidal in shape and were patterned after the test conditions conducted by Bazin and Saunier (see SHELL OIL CO. sections and Reference 64). It can also be observed that strain gages were mounted on each side of the specimen. Thus the tensile strain (initial) was recorded directly and even though constant stress testing was done, stiffness measurements were not necessary to compute tensile strains.

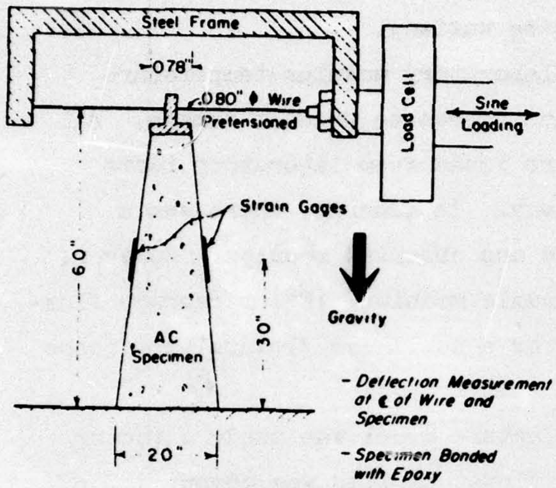


Figure 50. Typical laboratory fatigue array (from Coffman et al.[79])

The loading was applied at one 10-Hz haversine pulse per second. This is equivalent to a 0.1-sec load time and a 0.9-sec rest (dwell) time. Temperature was controlled by a methorial-water mixture around the sample from a constant temperature bath.

It was noted by Coffman that at low temperature (i.e. 6.5°F), failure (fracture) occurred very shortly after crack detectors (conductive paint) showed the first presence of crack formation. However, at temperatures above 205°F, cracks formed roughly at about one-half the repetitions required for complete fracture. Also of interest is the fact that the sample crack progression started at one side, then transferred to the other side and appeared to propagate toward the middle of the section until complete fracture was observed. The results of the fatigue tests are illustrated in Figure 51. It is to be especially observed that the effect of temperature upon the fatigue-life relationship is very dramatic, and, thus, results by Coffman obviously contradict the "strain criterion" advocated by Pell and previously described in this report.

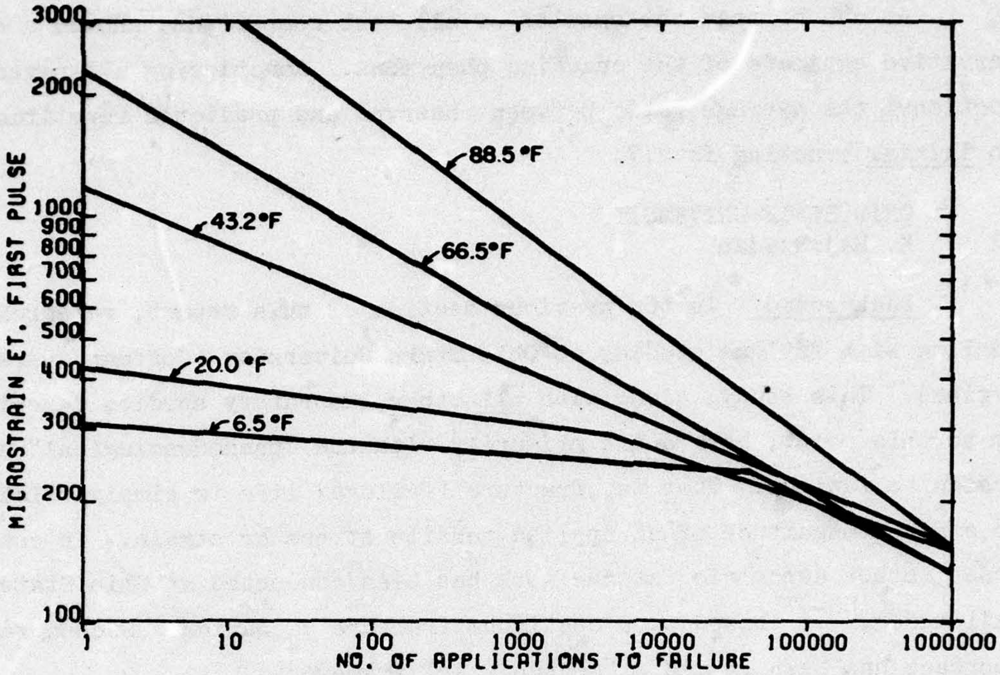


Figure 51. Results of laboratory fatigue tests (from Coffman et al.[79])

Using the results of the laboratory fatigue data shown in Figure 51, along with a multilayered analysis of each pavement test slab, predicted repetitions to fatigue failure were determined. These predictions in turn were compared with observed repetitions to initial cracking for the slab sections subjected to the repeated load testing. The results of this comparison are shown in Table 4.

Table 4

Repetitions to Observed and Theoretical Pavement Cracking

AC Thickness in.	First Observed Cracking	First Theoretical Cracking	Observed/Predicted Ratio
2.7	125,600	36,000	3.49
3.0	5,220,000	2,250,000	2.32
5.0	1,903,000	570,000	3.34
6.5	494,000	258,000	1.73

As can be seen the results of all test comparisons showed a conservative estimate of the cracking phenomena. Considering all pavement sections, the average ratio between observed and predicted repetitions to <u>initial</u> cracking is 2.7.

OHIO STATE UNIVERSITY
K. Majidzadeh

<u>Background.</u> In the previous section of this report, research dealing with fatigue studies at Ohio State University (Coffman) were described. This study, along with <u>all</u> other laboratory studies described up to this point, have dealt primarily with the "phenomenological" approach to fatigue. That is, fracture (failure) life is simply related to a given magnitude of an applied tensile stress or strain. In contrast, other asphaltic fatigue work has been conducted at Ohio State by Majidzadeh,[80-87] based upon continuum fracture mechanics. Such a fatigue approach has been termed a "mechanistic" framework.

Relative to "phenomenological" research studies, the use of fracture mechanics with asphalt fatigue research is relatively youthful, initiating in the very late 1960's. The major researcher at Ohio State is K. Majidzadeh but other associates (e.g. Kauffman,[82,83,85,87] Ramsamooj,[80,82-86] Chan,[80] and Chang[87]) have likewise assisted in the research studies. Salam and Monismith,[20,23] at the University of California, have also done correlative work between the two fatigue approaches based upon the concepts developed at Ohio State University.

<u>General Concepts.</u> The basic concept of the fracture mechanics approach to fatigue failure is built upon the hypothesis that asphalt concrete (like many other materials) possesses inherent flaws on its tension face before the application of an externally applied load. Upon subsequent load application, these "flaws" grow in size according to a defined crack propagation law. Once the size of the crack reaches a critical level, complete (spontaneous) fracture of the material occurs. Thus, there are three important stages within the mechanistic concept: initiation, propagation, and failure.

Mechanistically, the process is directly related to the energy

balance associated at the tip of the crack for discontinuity. In essence, the external work applied is equated to stored elastic energy, energy necessary for irreversible change in the material due to viscous or plastic flow and the energy required for crack formation. Because of the presence of the crack or flaw, stress concentrations are present near the crack tip. These concentrations cause a progressive sequence of crack resharpening and blunting (i.e. propagation) upon cyclic load application. Although the fracture (crack) process is discontinuous, the assumption of continuous growth of cracks is made in fatigue work.

Functionally, the repetitions to failure (fracture), N_f, may be denoted by:

$$N_f = f(I_c, R_c, F_c)$$

where I_c symbolically relates to the existence and characteristics of the initial or "starter flaw," R_c refers to the rate at which crack propagation occurs, and F_c denotes some critical condition at which R_c becomes infinitely large, that is, spontaneous fracture.

In mechanistic terminology, the previous functional equation is stated as:

$$N_f = f[(c_o)(A, n, K)(K_c)]$$

In this equation, c_o is the depth of the "starter flaw" from which the crack propagates. The rate of crack propagation term is seen to be dependent upon three parameters. The variables A and n are constants of the material and test conditions. The K value is termed the stress intensity factor and is a measure of the stress field in the vicinity of the crack in accordance with the load, size of crack, and geometrical and boundary conditions. It is also proportional to the force that causes the crack extension.[85]

These three variables are related to the crack rate,

$$R_c = \frac{dc}{dN} \text{ by Paris' Law which states}$$

$$\frac{dc}{dN} = AK^n$$

The last term, K_c, is termed the critical stress intensity factor. Thus it can be observed that $K(K_c)$ is a very significant parameter as it describes not only the stress field for various conditions but also defines the failure level or criteria for fatigue. The K_c value is a unique materials property for a given temperature. Evaluation of these parameters (e.g., c_o, A,n,K, K_c) allows the fracture life, N_f, to be theoretically determined from integration of the Paris equation between the limits of initial and critical conditions.

Obviously, the general approach outlined potentially affords several distinct advantages over the phenomenological approach. This is directly due to the ability of the mechanistic concept to include (directly incorporate) mode of loading effects (i.e. constant stress or constant strain). In addition, it directly recognizes and accounts for effects due to crack propagation and stress redistribution. As such, it provides a potentially powerful tool for future inclusion within a completely rational framework for structural pavement analysis.

LABORATORY AND LITERATURE
RESULTS

A. General. Almost all of the fracture and fatigue tests conducted at Ohio State University have been accomplished with sand-asphalt materials at ambient (room) temperature. A limited number of tests have been done on asphalt concrete specimens as well as at temperatures of about 20°F and 45°F. The organized research program initiated with fatigue (mechanistic) studies of simply supported beams, progressing to a beam on an elastic foundation followed by studies of slab behavior on elastic foundations.

B. Critical Stress Intensity Factor-- K_c. The K_c or critical stress intensity factor represents the level of K at which spontaneous fracture of the material occurs. This value, for a given material, is normally evaluated from fracture toughness tests. The K_c value is a distinct materials property which is independent of the crack to

thickness ratio of the specimen. However, it is a function of temperature for asphaltic materials and also demonstrates a slight dependency upon load rate. Thus, in order to evaluate this parameter for use in an analytical model which considers the total environment and load variables, K_c values for each material would have to be experimentally determined as functions of temperature and frequency of load.

It is important to recognize that the use of the theory is applicable only for plane strain conditions. For plane stress boundary values, the K_c term is no longer a material's constant. In order to satisfy plane strain conditions, the following limitations are made for two load cases:

1. $d > 2.5\left(\frac{K_{1c}}{\sigma_y}\right)^2$ Beam on elastic foundation

2. $h > 1.25\left(\frac{K_{1c}}{\sigma_y}\right)^2$. Slab on elastic foundation

In these equations, d is the beam depth, h the slab thickness, and σ_y the yield strength at the same load conditions as the fatigue test.

From these equations combined with the analysis of laboratory tests and typical flexible pavement constructions, the following important observations have been made. For thick pavement systems or specimens tested at low temperatures, fracture is governed by the K_c term and hence the critical crack size c_f is a fraction of the pavement or specimen thickness. For thin pavements or where specimens are tested at elevated temperatures, fracture is not governed by the K_c value. This may be interpreted by stating that the c_f value under these conditions is greater than the specimen or pavement thickness. Thus spontaneous fracture has been shown to occur for only thick sections and/or low temperature conditions. For other conditions, failure, as defined by the mechanistic approach, will not occur. These results have prompted Majidzadeh to assume that actual asphalt cracking failure may be defined

by either the c_f value as determined from the K_c value (thick pavements--low temperatures) or when the percentage of cracking (longitudinal extent) is 10 percent (thin pavements--high temperatures). It has been further assumed that in the N_f equation, c_f be set equal to the layer thickness for conditions where the K_c is not applicable.

The K_c value for any test conditions is evaluated experimentally from fracture tests on simply supported beams. The critical K_c value may be obtained using either the Winne-Wundt equation or the Srawley equation. The Winne-Wundt equation is:

$$K_c^2 = \sigma_n^2(1 - \mu^2)h\left[f\left(\frac{c_f}{d}\right)\right]$$

where

$\sigma_n = P_f l/4b(d - c)^2$

P_f = fracture load on specimen

c = crack depth

b = beam width

$h = d - c$

d = depth of beam

c_f = crack depth at the failure (fracture) condition

$f(c_f/d)$ = function given by Winne-Wundt in their fracture analysis study

The Srawley approach uses a dimensionless (square) stress intensity factor given by:

$$y^2 = \frac{16K^2d^3b^2}{P^2\ell^2}$$

The relationship (function) between c/d and y^2 is shown in Figure 52. The K_c value may be obtained by sawing a notch (crack) of given length, c, through a specimen and then finding the fracture load, P_f, on the simply supported beam. Knowing the c/d ratio and using Figure 52 (or the Winne-Wundt equation) the y^2 value may be determined. From y and P_f, the critical stress intensity factor K_c is:

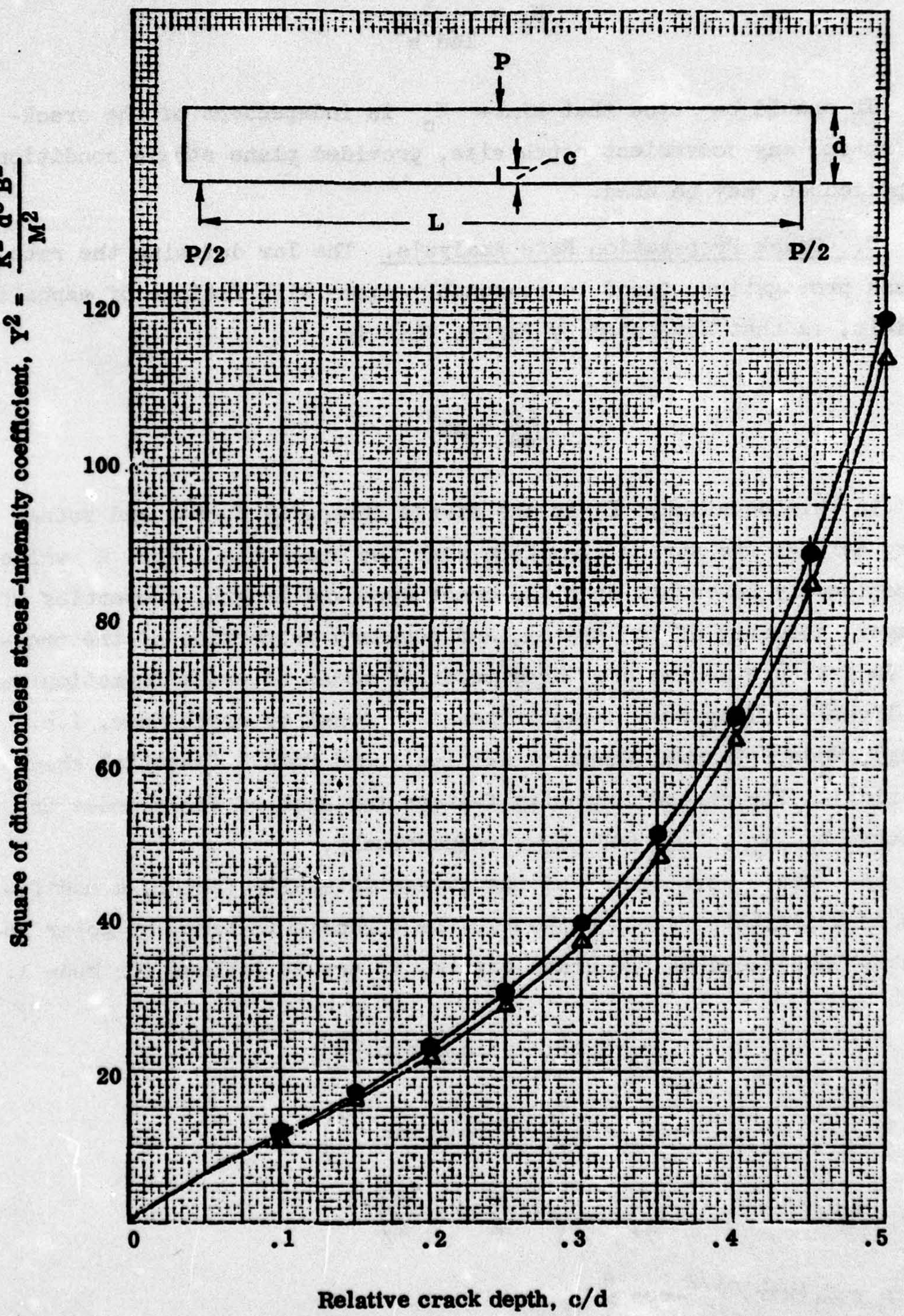


Figure 52. Dependence of square of stress intensity coefficient on relative crack depth

$$K_c = \frac{y^2 P_f^2 \ell^2}{16 d^3 b^2}$$

It should be noted that since K_c is independent of the crack-depth ratio, any convenient notch size, provided plane strain conditions are applicable, may be used.

C. Crack Propagation Rate Analysis. The law defining the rate of crack propagation, found to reasonably agree with fatigue of asphaltic materials, is that developed by Paris; that is

$$\frac{dc}{dN} = AK^n$$

In this equation, K is the stress intensity factor and subsequently defines the stress elevation near the crack tip. The K value is functionally dependent upon the crack type and length, properties of the layer, geometry of the system, and boundary properties of the problem. In fracture mechanics, three separate modes of crack formation may be applicable. They are: Mode 1 (tension normal to crack face, i.e. bending); Mode 2 (normal shear or sliding); and Mode 3 (parallel shear or tearing). The stress intensity factors for each of these modes are designated by K_1, K_2, and K_3, respectively.

In Irwin's development of the stress intensity factor, a modification of Griffith's fracture theory, the stresses in terms of polar coordinates introduced at the crack tip (r, θ) are as follows for Mode 1.

$$\sigma_x = \rho\left(1 - \sin\frac{\theta}{2}\sin\frac{3\theta}{2}\right)$$

$$\sigma_y = \rho\left(1 + \sin\frac{\theta}{2}\sin\frac{3\theta}{2}\right)$$

$$\tau_{xy} = \rho\left(\sin\frac{\theta}{2}\cos\frac{3\theta}{2}\right)$$

where $\rho = K_1(2\pi r)^{-1/2}\cos\frac{\theta}{2}$

for Mode 2 conditions

$$\sigma_x = \xi\left[-\sin\frac{\theta}{2}\left(2 + \cos\frac{\theta}{2}\cos\frac{3\theta}{2}\right)\right]$$

$$\sigma_y = \xi\left(\sin\frac{\theta}{2}\cos\frac{\theta}{2}\cos\frac{3\theta}{2}\right)$$

$$\tau_{xy} = \xi\left[\cos\frac{\theta}{2}\left(1 - \sin\frac{\theta}{2}\sin\frac{3\theta}{2}\right)\right]$$

where $\xi = K_2(2\pi r)^{-1/2}$

It is important to recognize that because K_i is dependent upon the functions previously noted, unique K_i functions result for different test procedures (e.g. simply supported beam, beam on elastic foundation, slab on elastic foundation). As a result, several general methods are available in which the stress intensity factor may be found. They may be generalized into theoretical solutions (Boundary Collocation and Finite Element) and experimentally derived functions.

For simply supported beams, the equations previously defined for the critical stress intensity factor K_c (i.e. Winne and Wundt's equation or Srawley's equation) can be used. It should be noted that for this test procedure, Mode 1 is applicable and the corresponding K term is found. Other theoretical methods for beams on an elastic foundation as well as semi-infinite cracks, radial symmetric cracks existing within a slab, can be found in References 82, 84, and 87. Several computer programs are available for these complex solutions.[87]

Ramsamooj has noted however that for a bonded slab conditon, even the most current refined theoretical solution differs from observed experimental data.[85] As a consequence, it appears that the simplest way to ascertain the relationship is by Irwin's equation relating the change in compliance, L, to the K value. For beams, the equation is:[80]

$$\frac{\partial L}{\partial C} = \frac{2}{E}(1 - \mu^2)\frac{K^2}{P^2}$$

while for slabs

$$\frac{\partial L}{\partial (2c)} = \frac{2}{E} (1 - \mu^2) h \frac{K^2}{P^2}$$

The principle of this procedure is that the change in deflection (increase) is uniquely related to the increase in accompanying crack size due to repeated load applications. As the compliance, L , is simply the reciprocal of the load-deflection relationship, the stress intensity function as well as the rate of crack propagation for any given boundary value may be obtained indirectly from deflection-repetition experimentation. To illustrate the use of this approach, the procedure for a simply supported beam is considered.

By making predefined notch lengths in several beams and then obtaining the load-deflection relationship, the compliance, L , may be determined for each test. By normalizing the compliance (L/L_o), a functional relationship between this parameter and c (crack depth) may be obtained as shown in Figure 53. Using unnotched specimens, a fatigue test is next conducted and the normalized compliance (L/L_o) as a function of load repetitions obtained. This is also shown in Figure 53. As a result, the relationship between c and N_f is now developed as shown in Figure 54. This function can be numerically differentiated to obtain the $\partial c/\partial N$ versus c relationship. Using Irwin's equation along with the results of the fracture test (L/L_o versus c), the K value can be obtained for given c and P values. Thus the relationships between $\partial c/\partial N$ versus c can be transformed into data pairs of $\partial c/\partial N$ versus K . These data when plotted on a log-log plot then define the crack propagation rate law in the form of:

$$R_c = AK^n$$

A typical result of laboratory tests is shown in Figure 55. From this result, the parameters A and n can be defined. It has been found that for fine-grained asphalt mixtures the value of n is approximately 4 for temperatures between 20°F and 80°F (slabs on

AD-A035 873 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISS F/G 13/2
PAVEMENT PERFORMANCE MODELS. VOLUME I. REPEATED LOAD FRACTURE O--ETC(U)
AUG 76 M W WITCZAK DOT-FA73WAI-377
UNCLASSIFIED WES-CR-S-76-15-VOL-1 FAA-RD-75-227-1 NL

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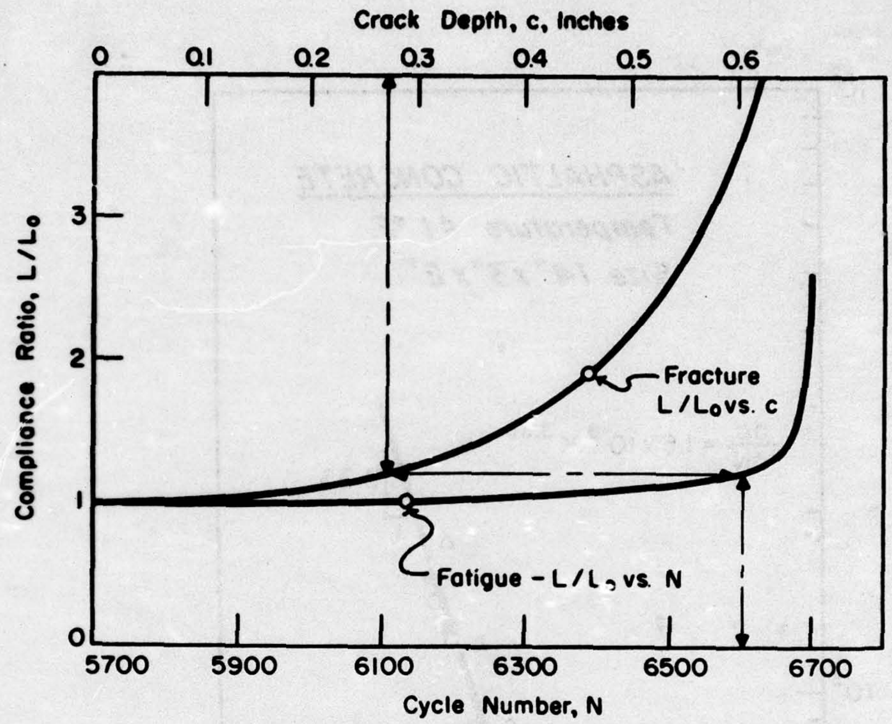


Figure 53. Normalized compliance versus N and c from fatigue and fracture tests

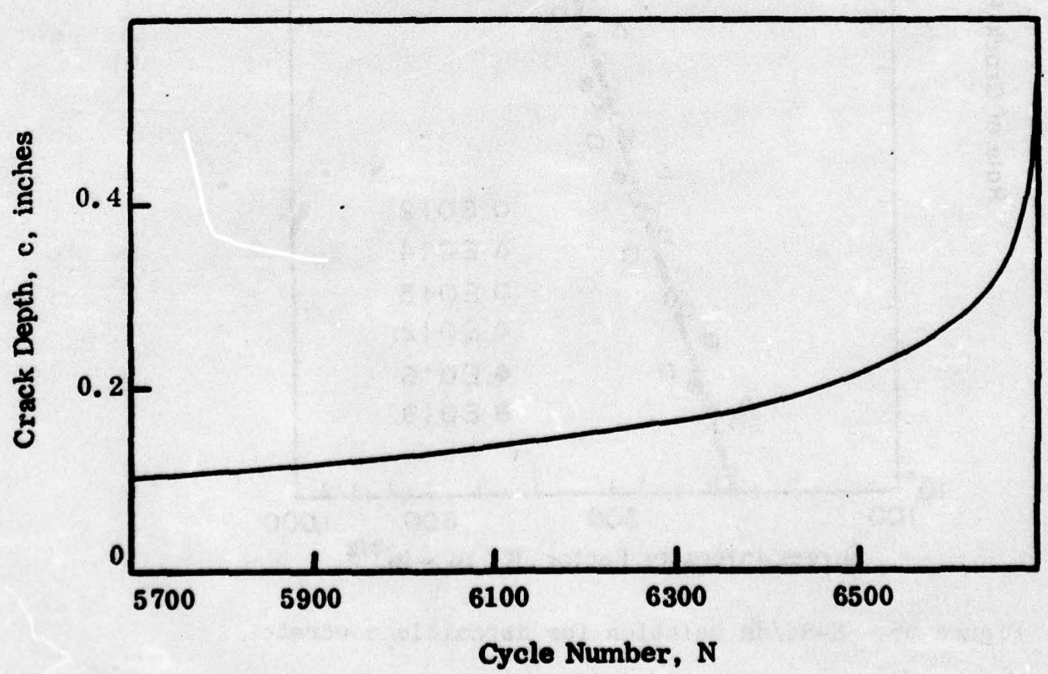


Figure 54. Typical c versus N curve for fatigue test

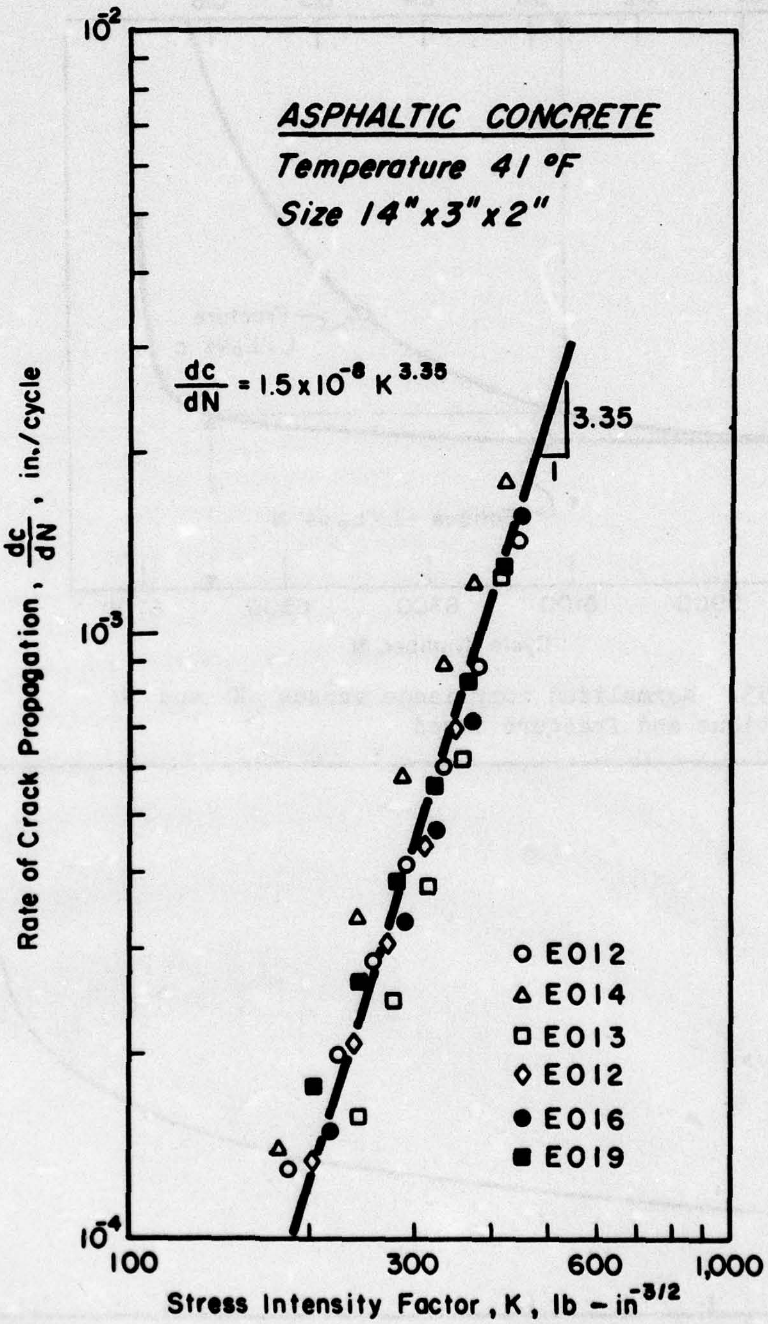


Figure 55. K-dc/dn relation for asphaltic concrete

elastic foundation). For asphalt concretes the value of n has been determined to be about 2.5 to 2.75.[87] The value of A has likewise ranged between 10^{-10} and 10^{-14} with the former being applicable for asphalt concrete and the latter being typical of finer grained asphaltic mixes (sand asphalts). One final, but important conclusion, is the fact that identical relationships are obtained for the crack rate law when using a beam on an elastic foundation as well as a slab on an elastic foundation. This implies that beam studies may suffice as laboratory tests to evaluate the A and n constants. The parameter A is functionally dependent only upon the test temperature and load conditions for a given material. Because of creep effects that may be associated with high temperature tests on simply supported beams, it has been recommended that an elastic foundation be used with the beam tests to evaluate these constants.[82,83,86,87]

D. Starter Flaw. The initial or "starter flaw" necessary for crack propagation cannot be directly measured. Instead it may be computed from experimental data. Although the c_o value is a materials constant, it is subject to statistical variation, and this has been hypothesized to account for the well documented statistical variation of all fatigue data. The magnitude of the starter flaw, c_o , can be found by extrapolating the crack growth law back to $N = 1$. Thus

$$c_o = c_f - \int_1^{N_f} AK^n dN$$

The solution of this equation is complicated by the fact that K is a function of c . As a result, numeric techniques with a computer program to determine c_o have been developed.[80,85-87]

Based upon this procedure, it has been found that for sand-asphalt beam tests, c_o values average about 0.05 to 0.08 in. while for asphalt concrete beam tests an average nearer 0.025 in. was found. However, when slab studies were investigated with the same materials, it was found that the average c_o values were about 2.5 to 3 times that determined from the beam tests. It has been stated that for typical pavement

analysis, fatigue tests on either slabs or 3-dimensional beam configurations may be necessary.[84] In any event, typical c_o values (for slab conditions) are about 0.20 in. to 0.25 in. for fine-grained mixes and 0.20 in. to 0.50 in. for asphaltic concrete mixes.

Verification and Design Application. In general, verification of the mechanistic approach to asphaltic concrete fatigue advocated in this section has been limited to comparison of predicted behavior (from the mechanistic concept) with observed behavior from laboratory results for a sand-asphalt mix (beam or elastic foundation) at 23°F and the results of the 6.5-in. full-depth slab discussed previously in the last section (see B. S. Coffman--Ohio State University).

Using average values of c_o , A , n , and K_{1c} generated from the beam studies, a predicted load-repetition plot was generated in accordance with the proposed theory. The comparison is shown in Figure 56.

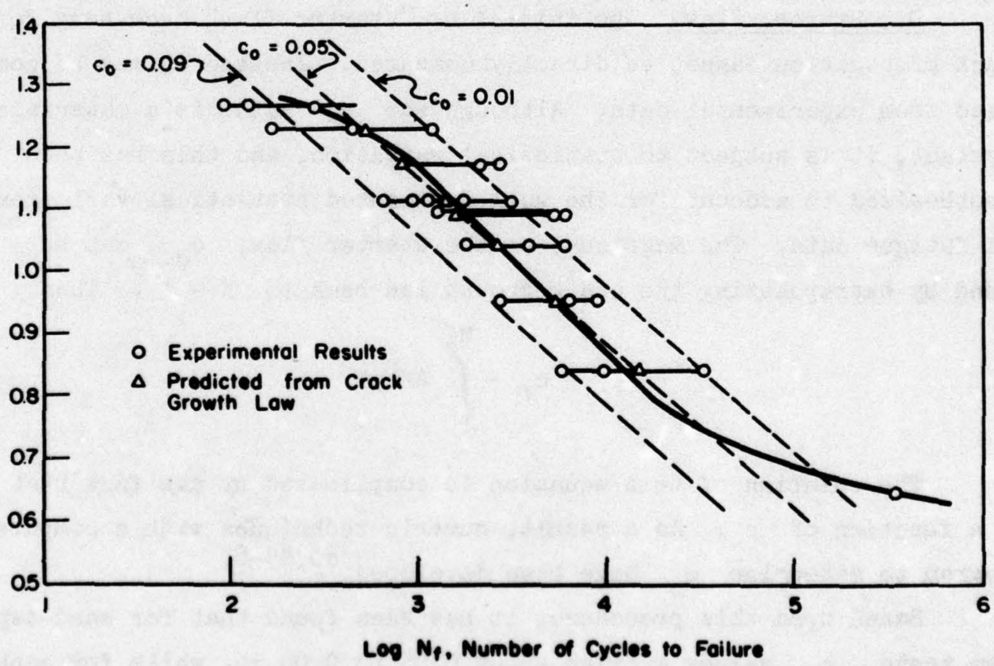


Figure 56. Relation between P_{max} and N_f (from Majidzadeh[87])

As can be seen, the agreement is quite good. An equally good comparison was also obtained with the slab study. From Coffman's results, it was

estimated that initial (surface) cracking appeared after 100,000 to 150,000 load cycles. Using fracture concepts for a slab analysis with typical fracture parameter values and a value of $c = h$ (slab thickness), it was estimated that 105,000 load applications would be required from the theory.[87] There have been no other verifications of the system for either laboratory, prototype slab tests or with actual pavement performance studies.

There currently exists no accepted design method using these concepts. However, a detailed outline of a mechanistic design procedure has been suggested.[85-87] The procedure is quite complex and further details may be found in the quoted references.

In general, the design concept is formulated upon fractures and fatigue tests using a "beam on an elastic foundation" approach. A procedure is presented whereby load equivalency factors may be obtained for various single and tandem axles based upon the rises and falls of the stress intensity factor distribution along the axis of vehicle movement. The damage model uses the K_1 and K_2 modes of crack formation and is given by

$$\frac{dC}{dN} = A_1 K_1^{n_1} + A_2 K_2^{n_2}$$

where the stress intensity factors are the average values of the rises and falls of the K factors obtained from the load-time (traffic mix) history.[87]

An incremental damage concept is used for each passage of the load train. The damage increment is given by

$$\Delta c = \sum_{p=1}^{n} \left[A_1 \left(\Delta K_{1p}\right)^{n_1} + A_2 \left(\Delta K_{2p}\right)^{n_2} \right]$$

Thus, an incremental solution is conducted whereby the length of the crack is increased, K_1 and K_2 values are recomputed, and the values are checked against the critical values (K_{1c} and K_{2c}). As previously

noted, failure may occur either when $K_1 \geq K_{1c}$ ($K_2 \geq K_{2c}$) or when the percent cracking in a longitudinal direction exceeds a certain value (e.g., 10 percent). It should also be pointed out that such an analysis is conducted on a trial pavement structure. Thus, if the results are unsatisfactory, a new thickness (structure) is assumed and the process repeated.

EMULSIONS (ASPHALT AND CEMENT MODIFIED), CHEVRON ASPHALT COMPANY

BACKGROUND

For the past several years, Chevron Asphalt Research has conducted fatigue related research dealing with asphaltic concretes, asphalt emulsions, and cement-modified emulsions.[88-94] Most of their fatigue work has concentrated on regular and modified emulsions with the primary objective of developing a complete structural design system for the materials using multilayered theory.[92-94] In addition, because of the dependence upon any fatigue computational design method upon materials (modulus) characterization, much allied research has been conducted with the use of the diametral M_R device introduced by Schmidt.

The basic laboratory fatigue procedures used by Chevron have relied upon constant stress testing with a pulse load of 0.1-sec duration along with a 0.5-sec rest time. This is equivalent to 100 applications per minute. Fatigue tests have been conducted with flexural beam tests comparable to that previously described in the University of California and TAI portions of this report. Additionally, mix stiffness (modulus) has been directly measured by deflections of the beam being fatigued.[90]

RESULTS--TYPICAL FATIGUE CURVES

Based upon the results of laboratory tests along with an interpretative analysis of other fatigue researchers, typical field fatigue curves for emulsions and cement-modified emulsions have been prepared by Chevron. The use of cement, as an emulsion modifier, is predicated primarily upon the great increase in the rate of strength (curing) observed with low cement contents (about 1 to 3 percent portland cement

with a nominal value of 1.3 percent being noted in the literature). This modification in material type was found to definitely improve the most probable serious drawback of emulsions, that is, the slow development of strength.

Figures 57 and 58 show the typical field fatigue criteria for emulsions and cement-modified emulsions. The curves shown have already been adjusted by a factor of 3 from laboratory data. This was done to approximate the differences between laboratory and field behavior. In addition, it can be seen that the curves are valid for air voids and asphalt volumes of $V_v = 5$ percent and $V_B = 11$ percent .

For other mixture characteristics, an adjustment is to be made to the N_f read off of the diagrams. The adjustment used is somewhat based upon Pell's factor of the ratio of bitumen volume to voids in the mineral aggregate. That is:

$$\frac{V_B}{V_v + V_B}$$

The equation used for correction is:

$$N_c = N_f 10^e$$

where

$$e = 4.84 \left(\frac{V_B}{V_v + V_B} - 0.69 \right)$$

Thus for a given tensile strain, N_f can be read off Figure 57 or 58 at a given stiffness condition. The design number of repetitions to failure N_c can then be computed from the equation to account for the particular mix volume proportions.

Finally, it should be observed by the reader that the fatigue curves shown in Figure 57 for asphalts and emulsions are the same as those proposed by C. L. Monismith and shown previously as his typical fatigue criterion. The only difference in the curves is that the shift factor of 3 has been applied to the Chevron results.

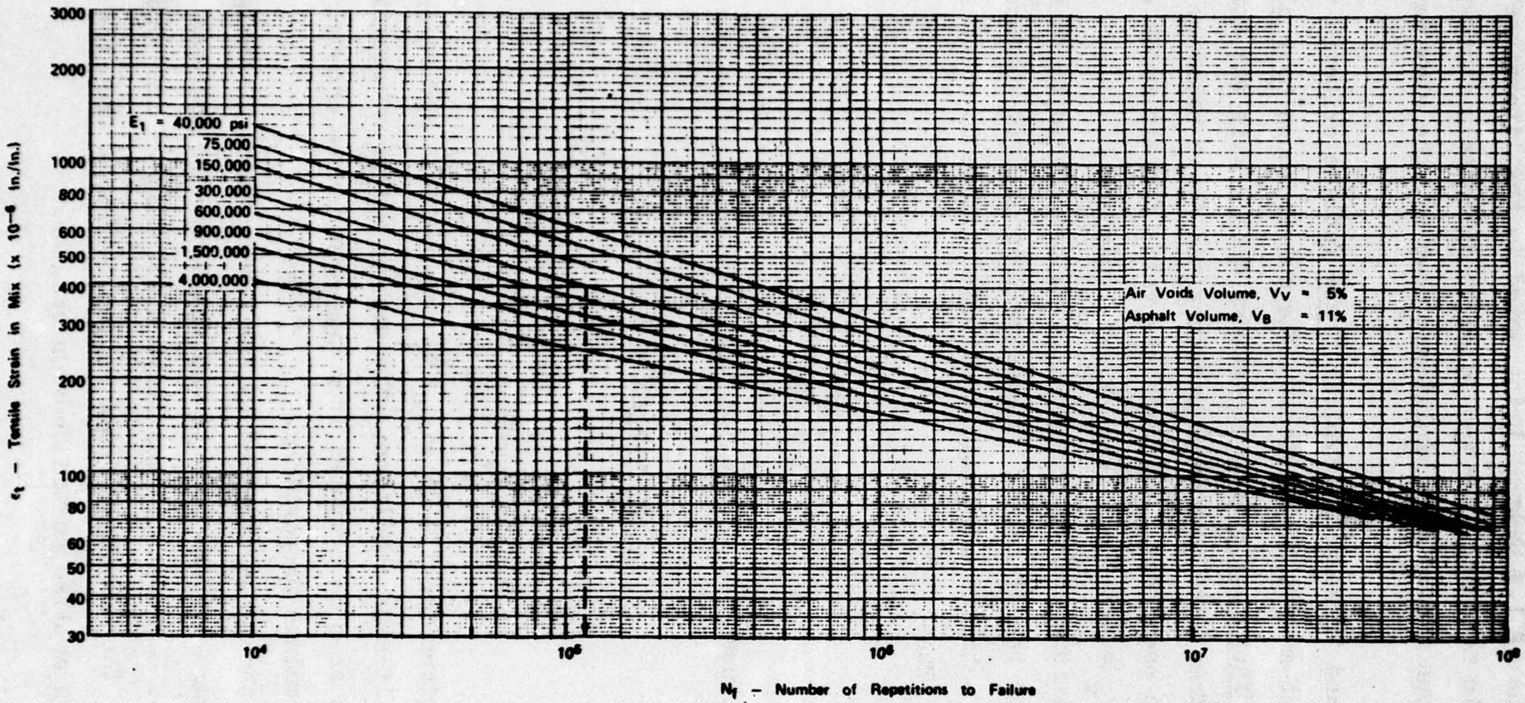


Figure 57. Field fatigue criteria for asphalt and emulsified asphalt mixes--field adjustment factor = 3 (from Chevron Asphalt Research Company[94])

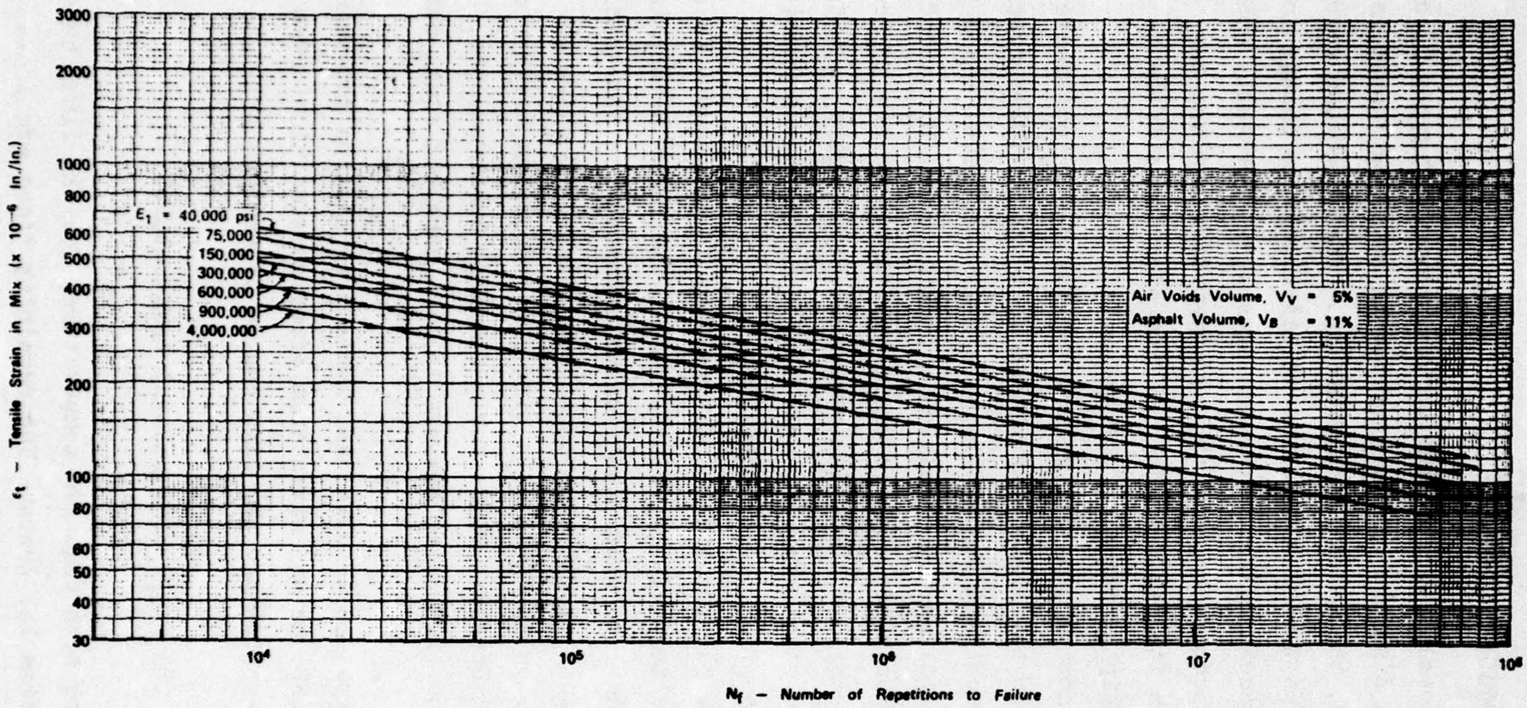


Figure 58. Field fatigue criteria for cement-modified emulsion asphalt mixes (C-EAM)--field adjustment factor = 3 (from Chevron Asphalt Research Company[94])

DESIGN APPLICATIONS

As noted previously, the fatigue curves shown in Figures 57 and 58 represent only one distress system that has to be investigated in the Chevron design method; the other is deformation. The procedure recommended by Chevron makes use of a direct cumulative fatigue damage model with the curves and accompanying correction equations previously noted. For analyzing asphalt concrete mixes and cement-modified mixes the procedure is direct in its application once the specific stiffness-temperature relationship is measured for the mix.

However, with normal emulsion mixes, special design considerations must be investigated to take into account the fact that a fully cured condition may not be achieved for several years in the field. As a result, two cure conditions must be analyzed: initial cure M_i and final cure M_f. For emulsion mixes, M_R (asphalt stiffness) at any ambient temperature after a 1-day air cure simulates the initial cure condition while a test condition of a 3-day air cure plus a 4-day vacuum dessication approximates the final cure condition.

Figure 59 shows how the effect of cure time is considered in establishing the modulus temperature relationship. The three data points on the figure represent laboratory test values obtained for the initial and final cure conditions previously described. Figure 60 shows the correlation between anticipated field cure time and evapotranspiration conditions for the United States. Thus, for a specific location, the anticipated cure period (i.e. time between initial cure and final cure) can be ascertained.

Once this information is obtained, Table 5 is used to determine the Reduction Factor (RF) at any given time after construction. The modulus at any given temperature and time can be determined by:

$$M = M_f - (M_f - M_i)(RF)(\text{constant temperature})$$

In Figure 59, it can be seen that two temperatures for the final cure period are necessary from lab tests. The remaining lines are drawn parallel to the final cure condition. It has been shown from laboratory

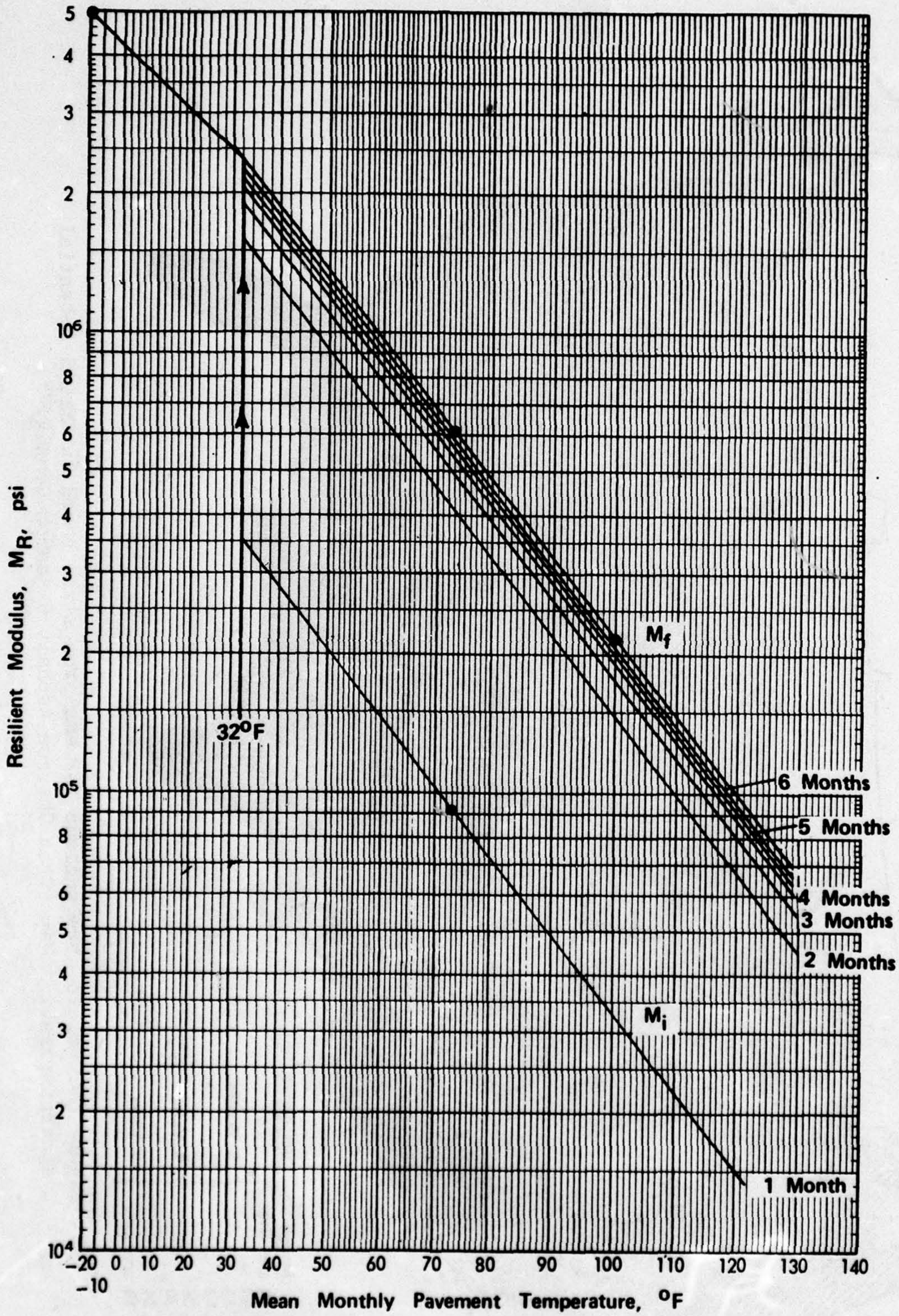


Figure 59. Typical modulus-temperature-cure conditions analysis for emulsified asphalt mixes (from Chevron Asphalt Research Company[94])

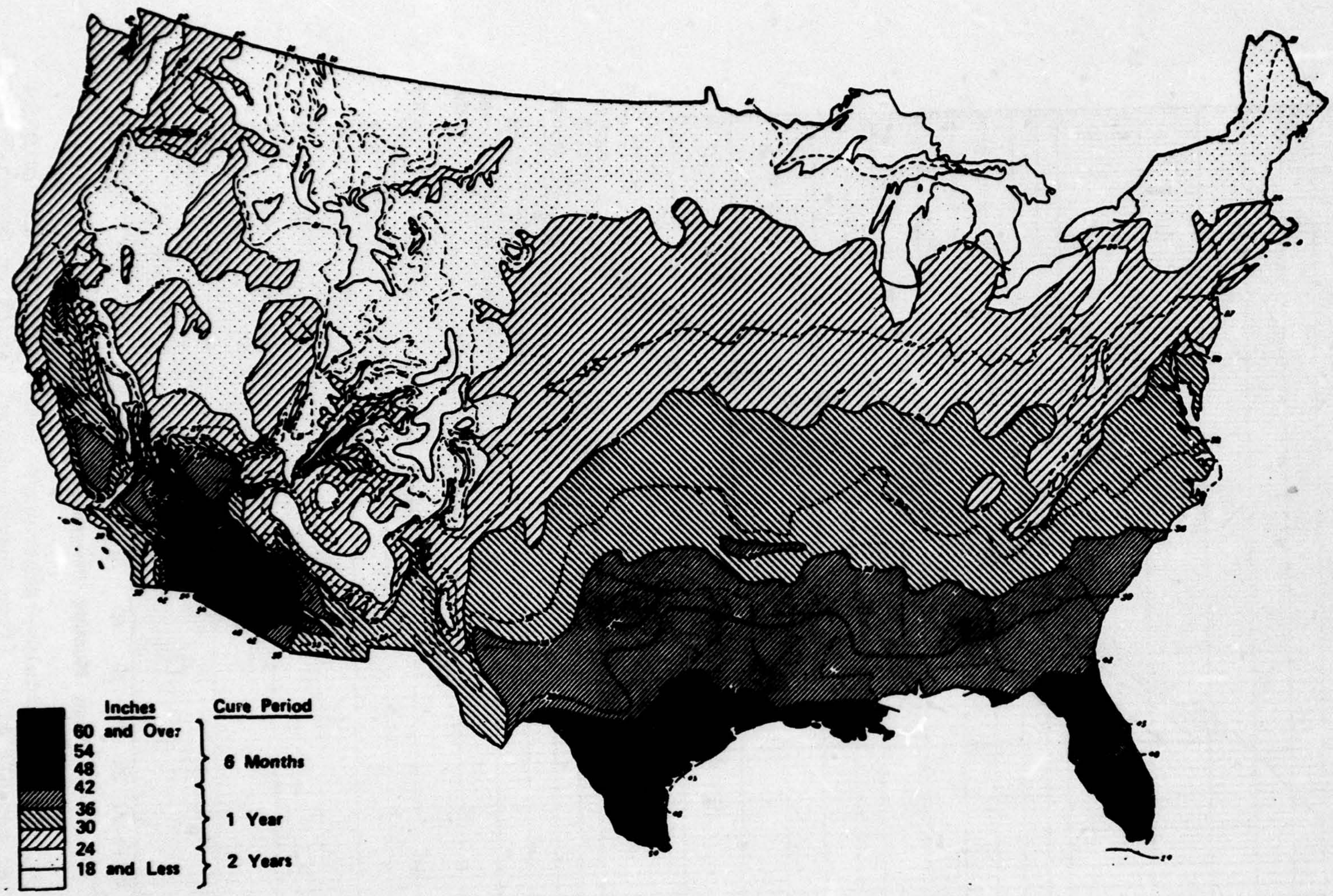


Figure 60. Field cure periods for emulsion mixes based on Annual Potential Evapotranspiration Map (from Chevron Asphalt Research Company[94])

Table 5

Cure Reduction Factor (RD) for Emulsion Mixes

Month	Reduction Factor 6-month Cure	1-yr Cure	2-yr Cure	Month	Reduction Factor 2-Yr Cure
1	1.0	1.0	1.0	13	0.198
2	0.37	0.62	0.78	14	0.175
3	0.225	0.48	0.69	15	0.154
4	0.136	0.37	0.62	16	0.136
5	0.082	0.29	0.545	17	0.120
6	0.05	0.225	0.48	18	0.105
7	--	0.175	0.42	19	0.093
8	--	0.136	0.37	20	0.082
9	--	0.105	0.33	21	0.073
10	--	0.082	0.29	22	0.064
11	--	0.064	0.255	23	0.057
12	--	0.05	0.225	24	0.05

work that this parallelism occurs with various degrees of water present in the mix (i. e. cure percentage).

This procedure now provides a complete time, temperature modulus relationship that can be introduced into a monthly cumulative fatigue damage model to predict the structural fatigue adequacy of a given pavement structure.

LIME-TREATED SOILS

The beneficial effects of lime-soil stabilization have been readily known from many years of pavement behavior and experience. In general, the addition of lime, particularly to clay soils, may serve as either a "modifier" and/or "strengthening" agent, depending upon the specific function deemed by the engineer. The addition of lime alters the plasticity characterization of the clay minerals and thus greatly alters the behavior of the treated soil. It should be understood that even though lime may not increase the strength of the soil (both compressive and flexural), the beneficial effects upon reduction of expansive characteristics, better field compaction properties, upgrading marginal materials, etc.... often are the principal reasons for this type of stabilization, and not strength increases.

However, when lime is used to increase the strength, the flexural behavior is also generally increased. Thus the ability to resist tensile stresses applied through repetitive loadings is enhanced. It should be realized that because of their general range in maximum strength, lime-treated soils are normally placed in lower (subbase) type pavement layers and therefore are not subjected to high tensile stresses due to the load.

Because of these factors, very little prior information concerning the fatigue behavior of lime-treated soils is available. In addition, the author is not aware of any current known structural design procedures using lime-treated soils that use a repetitive load type of analysis to design against fatigue crack initiation in this material layer. However, with the recent increased use of more "rational" or "functional" types of structural pavement analysis, the static tensile behavior,[95,96] fatigue response[97] as well as the dynamic modulus characterization[98] have

been made allowing a general analytical framework to be established for this material.

Work at the University of Texas and other agencies has shown that a great many factors affect the tensile strength and behavior of lime-treated soils.[95,96] Among these are the compactive type and effort; curing procedure, time, and temperature; molding moisture content; and lime and clay content. Studies have developed correlation models between tensile strength (static) with these aforementioned properties. Additionally, correlations between tensile strength and common strength tests, such as the unconfined compression and cohesion meter, have been determined.[95,96]

Thompson has shown that the stiffness of the lime-soil mixture can be related to the unconfined compression value. Common modular ratios between treated and untreated soils have been found to range between 3 and 25.[98] Results also indicate that after a reasonable cure period, the behavior is essentially elastic.

Fatigue testing of lime-treated soils has also been conducted at the University of Illinois by Swanson and Thompson.[97] Unlike fatigue results for asphaltic materials but common for results of pozzolanic admixtures, fatigue relationships are normally expressed in terms of a ratio of an applied stress to static flexural strength versus log repetitions plot. Figure 61 illustrates such a fatigue curve for a lime-treated soil obtained after a cure period of 30 days at 70°F.

Figure 62 shows several other fatigue plots for various lime-soil mixtures relative to typical curves for portland cement concrete (PCC) and lime-fly ash aggregate mixture. In general, it can be seen that most of the lime-soil fatigue behavior is much steeper than that of PCC. This implies that the fatigue response of the treated soil is less sensitive to changes in stress level.

No information was noted as to the existence of an endurance limit for these materials. However, it can be noted that for stress ratios near 50 percent, failure repetitions generally exceed 10^6 or more applications. There is also no direct information relating the correlation of laboratory fatigue results to field pavement behavior with these materials.

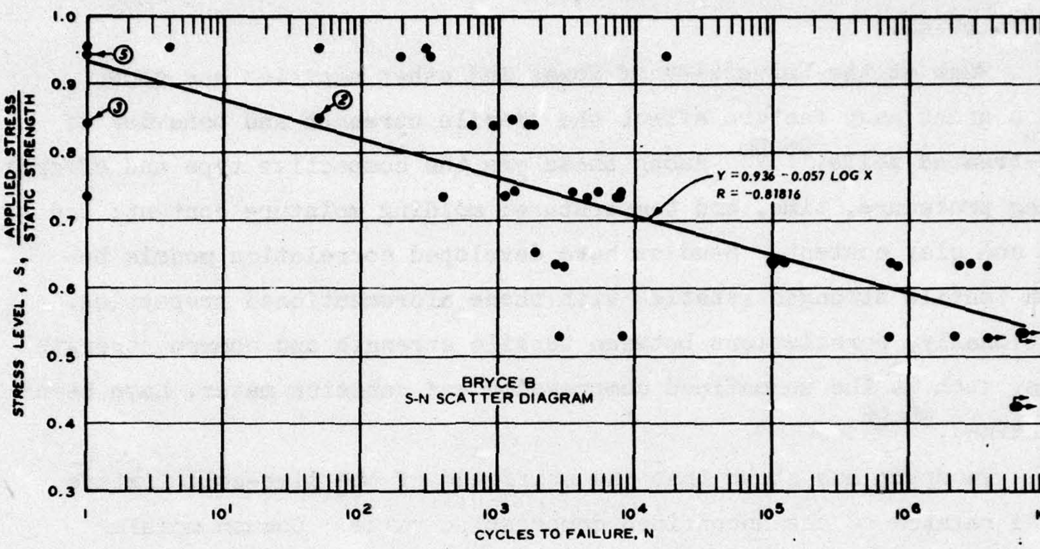


Figure 61. Typical lime-soil fatigue test results (from Swanson and Thompson[97])

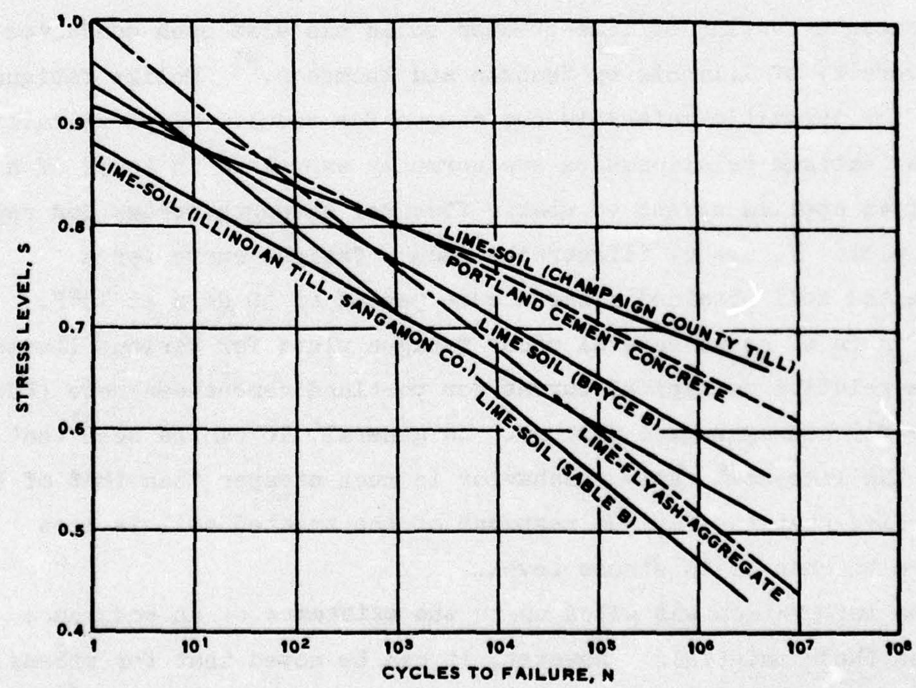


Figure 62. Comparison of fatigue response of lime-treated soils to PCC and lime-fly ash mix (from Swanson and Thompson[97])

LIME-FLY ASH AND LIME-CEMENT-FLY ASH MIXTURES

BACKGROUND

The recent need for using low quality materials of construction in the pavement industry has resulted in much research devoted to the use of various stabilizing agents. One such stabilizing ingredient, economically available in several portions of the country, is fly ash. Fly ash, when properly blended with hydrated lime, basically forms cementitious compounds of calcium silicates, identical with those in normal portland cement. This "pozzolanic" action provides a great increase in stiffness and strength to untreated aggregate materials. Sometimes, small quantities of portland cement are added to the lime-fly ash (LFA) mix and a lime-cement-fly ash (LCFA) mix is obtained. It is interesting to note that the function of the cement in an LCFA mix is primarily as an additive to accelerate the rate of development of the chemical bond. Thus, LCFA mix may be somewhat comparable to the cement-modified asphalt emulsions in that the introduction of small cement percentages acts to accelerate the strength gain.

One of the most important characteristics of obtaining proper strength gain is to allow proper curing conditions (time and temperature) to proceed after construction. Barenberg[99] has noted that below 40°F the chemical reaction for an LFA aggregate mix virtually stops. Above this temperature the rate of reaction increases with increasing temperature. Performance observations of several pavements using an LFA mix have demonstrated the importance of achieving a fairly high degree of cure before the first winter, especially in colder environments. A significant contribution has been made by MacMurdo and Barenberg[100] in developing a procedure based upon heat transfer theory and actual weather data (degree-days above a certain base temperature). This procedure allows the prediction of realistic cutoff dates for late-season construction in order to obtain a specified strength for typical LFA and LCFA mixes.

For LFA mixes, Yang[101] has noted the advantages of the relatively

long period of time (5 yr) required to achieve ultimate strength gain. As previously noted, the addition of cement normally results in a higher early strength gain for LCFA mixes. Barenberg[99] has stated that typical expected strengths of LFA mixes are 2000 to 3000 psi (5 yr) with some measured core strengths greater than 4000 psi being recorded.

The flexural strength (static) properties of LFA have been found to be about one-fourth of the compressive strength with ultimate tensile strains at fracture being about 200 to 400 μin./in. This value is somewhat comparable to fracture strains associated with plain concrete pavement.[99] In general, the material response is elastic but Barenberg has reported that for stresses beyond 60 to 70 percent of ultimate, the response becomes highly nonlinear.[99] The modulus of the LFA mix is obviously a function of the actual curing percentage. However, common values are between 1.5×10^6 and 2.5×10^6 psi. Poisson's ratio has been noted to be dependent upon stress level but a common value of 0.10 has been suggested.[99]

As with any pozzolanic reaction, the possibility of shrinkage cracks is always present. It appears, that as a general rule, the probability of obtaining shrinkage cracking is directly related to the rate of hydration. Barenberg has observed that some problems of shrinkage cracking have been observed with LFA aggregate materials.[99] However, he indicates that attention to compaction density requirements, at or below optimum moisture, will largely overcome this problem.

RESULTS--LABORATORY FATIGUE

Laboratory fatigue tests on both LFA and LCFA mixtures have been noted in the literature. Barenberg[99] has presented a typical fatigue relationship for an LFA mixture. This response is shown in Figure 63. Like the lime-soil fatigue responses previously shown, the fatigue relationship for pozzolanic type mixes is normally plotted as a straight line on an arithmetic stress to strength ratio versus logarithmic repetition plot.

Yang has reported that the fatigue relationship developed for the

LCFA mixture used at Newark International Airport was

$$\frac{\sigma n}{\sigma b} = 1 - 0.92 \log N_f$$

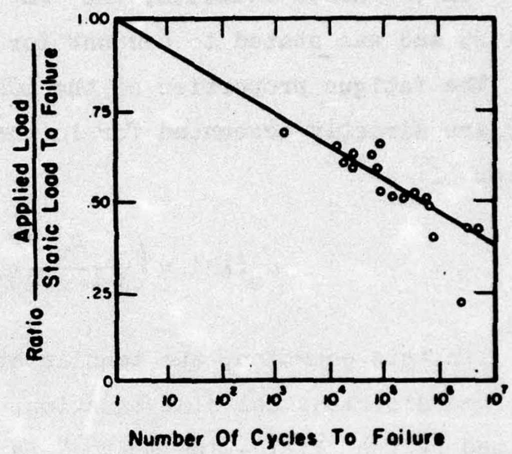


Figure 63. Laboratory fatigue relationship for LFA aggregate mix (from Barenberg[99])

where σn is the fatigue strength of the material at the n^{th} repetition of the traffic load, σb is the static bending (flexural) strength, and N_f represents the number of repetitions to failure.[101,102]

Both of these fatigue responses appear to have been obtained from direct laboratory tests. Neither researcher indicated, however, for which stage of curing the results were typical.

DESIGN APPLICATIONS AND VERIFICATION

A very detailed design procedure for pavements using LCFA aggregate mixtures has been developed by Yang[101] and the specifics of this analysis may be found in the noted reference. The overall design is quite complex and uses several design subsystems that must be considered. Relative to the analysis developed by Yang to insure against cracking of the LCFA mix, a working stress concept rather than a cumulative fatigue type of analysis is advocated.

In general, the flexural strength of the LCFA mix must be greater than the combined stresses due to the applied wheel load, differential settlement, and seasonal/daily effects of temperature (warping) σ_t .

Thus

$$\sigma_f \geq \sigma_e(FS) + \sigma_d + \sigma_t$$

In the above equation, the FS value used by Yang at Newark was 1.25 and was stated to account for material variability of the LCFA mix. The fatigue properties of the LCFA (previously noted in equation form) are directly accounted for in the load stress σ_e by the introduction of:

$$\sigma_e(FS) = \left(\frac{\sigma_w \text{ or } \sigma_y}{1 - 0.092 \log N}\right)1.25$$

In this equation, the tensile stress obtained from either Westergaard's classical slab equation, σ_w , or the yield stress, σ_y , obtained by the "Yield-Line Method" is used (see Reference 101 for further details). As σ_d and σ_t are nonload-related stresses, a design pavement thickness of the LCFA mix can be developed for any critical aircraft-repetition level desired.

Barenberg and associates at the University of Illinois have presented a comprehensive analysis of LFA aggregate mix pavement performance.[99] The study included laboratory testing to evaluate salient material characteristics, performance observations from static and dynamic wheel load testing at a laboratory test track, and qualitative evaluations of actual existing LFA pavements throughout the country.

The fatigue curve of the LFA mix used in the test track study has previously been shown in Figure 63. The difficulty in applying this fatigue curve directly to analytical procedures for fatigue fracture has been stated by Barenberg.[99] In essence, because the LFA materials increase in strength with time (recall that the period may be up to 5 yr in duration), corrections must be continually made to adjust for the strength gain during the load interval.

An additional and important result obtained by a comparison of predicted fatigue behavior from Figure 63 with observed failure repetitions of various pavement test sections relates to the type of theory

used to predict the stress state in the LFA pavement. It was concluded that a much better agreement resulted when Meyerhoff's ultimate load capacity theory was used rather than Westergaard's elastic slab analysis. Meyerhoff's theory is:

$$P_o = \frac{(4 + \pi) f_b h^2}{6\left(1 - \frac{2a}{3\ell}\right)} \quad \text{for} \quad \frac{a}{\ell} > 0.2$$

where P_o is the ultimate load in pounds, f_b is the modulus of rupture for the LFA in psi, h is the slab thickness in inches, a is radius of the equivalent circular loaded area, and ℓ is the radius of relative stiffness.

The more accurate comparison between predicted and observed failure repetitions for Meyerhoff's theory is shown in Table 6. This table, as can be seen, did not account for increases in strength with time for the LFA mix. Subsequent calculations, not presented in this report, demonstrated that a much closer agreement between predicted and observed behavior occurred when this strength gain was taken into account within the analytical framework.[99] Barenberg also noted from the evaluation of existing LFA pavements that good performance was observed on all sections where the ultimate strength of the LFA mix under edge load conditions was 1.5 to 2.0 times the applied wheel load.

CEMENT-TREATED MATERIALS

BACKGROUND

As referred to in this report, cement-treated materials used as stabilized layers, in pavements, collectively refer to three basic types of materials commonly used with portland cement. They are "soil cement," "cement-treated granular materials," and "lean concrete." All of these materials have been successfully used in construction throughout the world. However, it appears that use of soil cement has been much more widespread in the United States than abroad. In contrast, "lean concrete" is widely used, for example, in Great Britain, whereas in the

Table 6

Comparison of Predicted and Observed Failures for LFA Bases (from Barenberg[99])

Test Set	Test Pavement No.	Pavement Thickness in.	Modulus of Rupture at Time of Initial Loading psi	Applied Load lb	Theoretical Stress by Elastic Theory, psi	Ultimate Load Carrying Capacity lb	Ratio of Theoretical Stress to Modulus of Rupture	Ratio of Applied Load to Ultimate Load	Expected Load Applications to Failure: Using Theoretical Stress to M_r Ratio	Expected Load Applications to Failure: Using Applied to Load Ratio	Loads Applied to First Observed Failure
I	1	4.3	185	1850	122	8,200	0.66	0.23	5,100	1×10^8	1×10^6*
	3	3.8	185	1850	150	6,000	0.81	0.31	120	7×10^7+**	1×10^6+
	4	4.8	185	1850	101	9,500	0.55	0.20	81,000	1×10^8	1×10^6+
	5	4.3	185	1850	122	8,200	0.66	0.23	5,100	1×10^8+	1×10^6+
I	1	4.3	260	3180	210	11,500	0.81	0.28	120	1×10^8	3×10^5+*
	3	3.8	260	3180	258	8,400	0.99	0.28	1	1×10^7	3×10^5
	4	4.8	260	3180	174	13,300	0.67	0.24	4,000	1×10^8	3×10^5+
	5	4.3	260	3180	210	11,500	0.81	0.28	120	1×10^8+	3×10^5+
II	1	4.3	75	3180	198	3,120	>>1	1.02	<<1	1	1.5×10^4
	2	4.8	75	3180	168	3,880	>>1	0.82	<<1	90	5×10^5
	3	5.3	75	3180	143	4,680	>>1	0.68	<<1	3,100	5×10^5+
	4	5.8	75	3180	122	5,620	>>1	0.56	<<1	63,000	5×10^5
	5	4.8	75	3180	168	3,880	>>1	0.82	<<1	90	1.5×10^4+
	6	5.3	75	3180	143	4,680	>>1	0.68	<<1	3,100	5×10^5+
III	1	4.0	76	3180	360	1,630	>>1	1.95	<<1	<1	<10
	2	4.0	170	3180	385	3,650	>>1	0.87	<<1	20	175
	3	4.0	230	3180	395	4,880	>1	0.65	<1	7,000	5×10^5+
	4	5.5	230	3180	230	9,050	1.00	0.35	1.1	1×10^7	5×10^5+
	5	5.5	170	3180	227	6,230	>1	0.51	<1	3×10^5	5×10^5+
	6	5.5	76	3180	213	3,030	>>1	1.05	<<1	≈1	80

* Pavements loaded with 1 million applications of the 1,850-lb load followed by 300,000 applications of the 3,180-lb load.
** Values with + sign showed no sign of failure at the termination of the loading program.

United States it appears that little if any work has used this type of stabilization.

As a general guide, the quality and strength characteristics of the three materials identified increase toward the "lean-concrete" type of stabilization. As a result, common pavement engineering has dictated that soil-cement layers are normally used as subbase or improved subgrade layers while lean concrete is appropriate for use as a high quality base material for heavily traveled pavements.

The increase in strength is obviously brought about by the hydration of the cement added to the unbound soil and/or aggregates. Thus, the pozzolanic action forming the cemetitious bond is comparable to that of normal PCC pavements. As a result many of the material properties as well as their actual behavior as pavement layers perform similarly to normal PCC pavements.

Figures 64 and 65 illustrate typical strengths and moduli properties of these cement-treated materials. Because of the increased strengths both compressive and, perhaps more importantly, flexural characteristics are altered. Thus the addition of the cement, while greatly increasing the modulus of the material, also increases the ability of the material to withstand tensile stresses due to repeated bending. Figure 64 shows that a direct relationship between increased flexural strength and compressive strength exists. This is quite important as many specifications, while imposing a limit on the compressive strength of the cement-treated material, are indirectly specifying a minimum strength in flexure or bending. Almost all rational types of analysis on pavement behavior with cement-treated materials have shown that bending (tensile) stresses are the most critical factor affecting pavement cracking due to traffic. Figure 65 indicates the probable range in elastic moduli as a function of flexural strength for the three types of cement stabilized materials. It should be noted that the moduli shown have been obtained from electrodynamic tests and as a result are probably somewhat higher than other reported values using static results.

Because the hydration process for all three types of materials continuously proceeds with time, properties such as strength are time

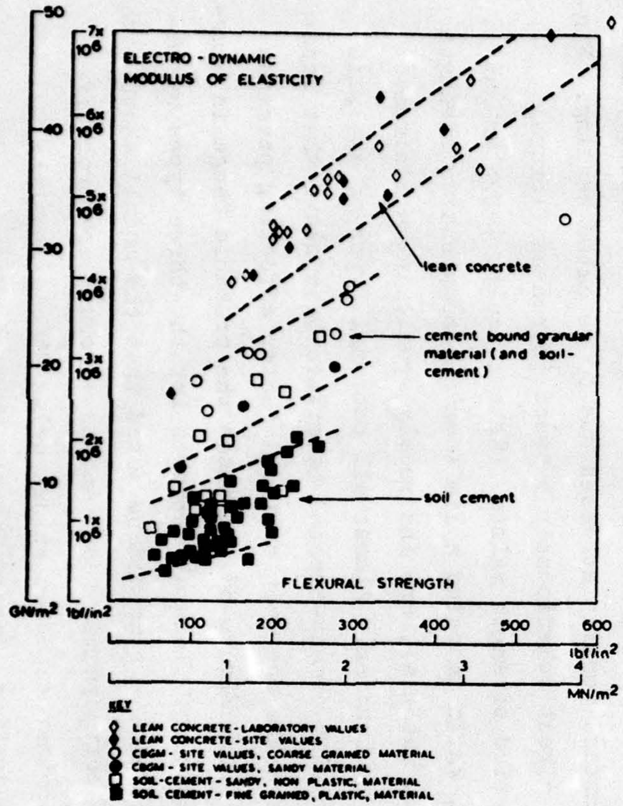


Figure 64. Compression and flexural strength correlations for cement-treated materials (from Lilley and Williams[103])

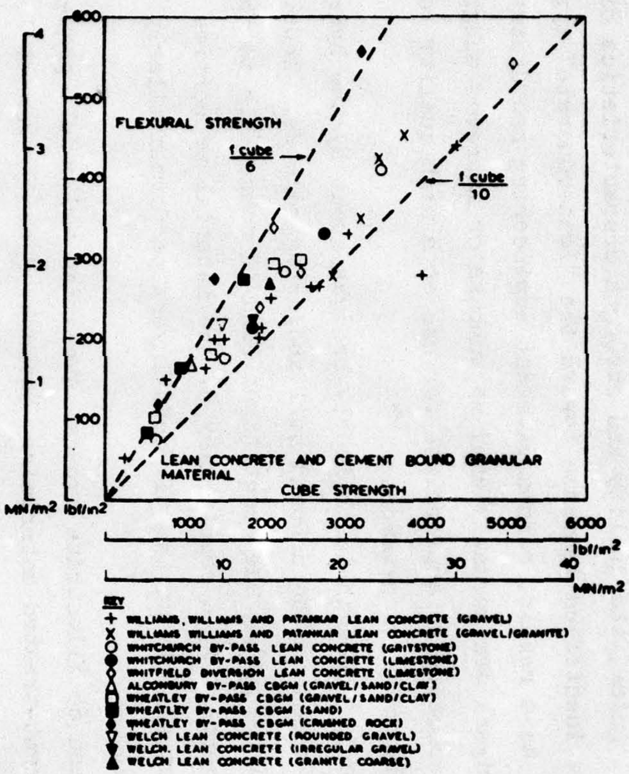


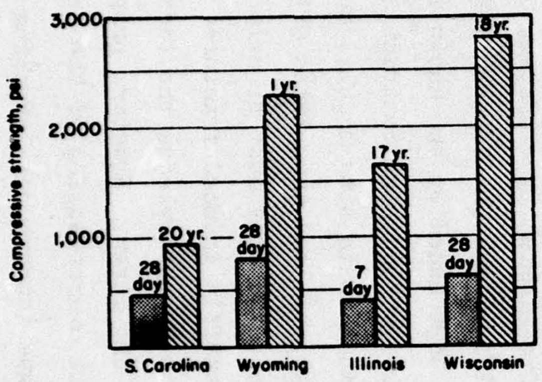
Figure 65. Typical modulus-flexural strength relationships for cement-treated materials (from Lilley and Williams[103])

dependent. It appears from published literature that the gain in strength for the soil-cement material occurs at a much slower rate over a long period of time compared with that of the lean concrete-normal PCC strength-age relationship. This is illustrated by Figure 66. Note that for the soil-cement figure strength percentages, relative to the 28-day strength, vary from 200 to 400 percent between 1 and 20 yr. However, such results are in conflict with other studies stating that 60 days result in fairly constant properties of soil cement.[105]

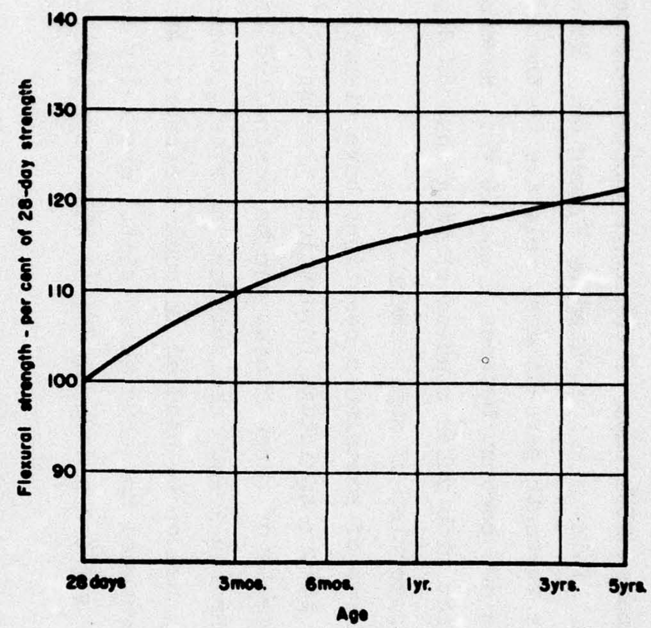
Extensive observations of pozzolanic reactions have clearly demonstrated that cracking due to the shrinkage (hydration) process will always occur. While the effects of these cracks can be controlled somewhat in normal PCC pavements through joints and reinforcing steel, cement-treated materials normally cannot be handled in such a manner. As a result, the treated layer designed to serve as a "slab," generally becomes cracked by shrinkage and thus loses some degree of slab action within the structure.

The occurrence of these cracks presents several analytical as well as practical problems to be overcome. First of all, the type of theory used to predict the stresses, strains, and deflections is obviously quite important and the applicability of multilayered systems, elastic slab analysis, and/or finite-element type systems capable of handling cracks (layer discontinuities) must be clearly defined. Secondly, the analytical problem is compounded by the known increase in laboratory properties with time acting in direct contrast to the decrease in field characteristics (such as in situ moduli) due to the progressive cracking from shrinkage and load associated effects.

Finally, a clear-cut design philosophy must be selected when using fatigue cracking analysis due to traffic induced fracture. In general, the prevailing design philosophy is to acknowledge the fact that shrinkage cracks will initially form in the layer. However, proper structural design is still required to prevent overstressing of the cement-treated layer to prevent "secondary" fatigue (structural) cracking due to repeated traffic loads.



(a) Soil Cement



(b) Portland Cement Concrete

Figure 66. Strength-time relationships (from Portland Cement Association (PCA)[104])

SHRINKAGE OF CEMENT-TREATED MATERIALS

The mechanism of shrinkage and shrinkage cracking is a complex phenomenon. It should be understood by the reader that high shrinkage does not necessarily imply high crack intensity. Rather the complex interaction between shrinkage stresses and tensile strength, at any particular moment, is the final criteria as to whether cracking will occur. Because of this, factors other than the properties (chemical) of the cement-treated layer enter into the picture. Considerations of layer geometry (e.g. thickness), friction restraint, and external environmental conditions such as mixing, curing, and temperature all are important external factors that must be evaluated in addition to the cement-layer properties.

Although a vast amount of research dealing with shrinkage mechanisms, chemical additions to reduce shrinkage cracking, theoretical models to predict shrinkage stress, etc., has been conducted, a general statement that all cement-treated layers will exhibit shrinkage cracking must still be made. Accordingly, current research appears to be focused upon (a) continued work on additional methods to minimize or eliminate shrinkage cracking, (b) construction methods and pavement designs to minimize or eliminate the reflective cracking through the upper (normally asphaltic) layer, and (c) development of "rational" or theoretical studies to determine stress state in pavements, presumed to have cracking present, and hence to be more susceptible to load-induced stresses brought about by the reduction in slab action due to shrinkage and reflective cracking.

Wang[106] has noted that for soil cement the major causes of shrinkage are due to: (a) loss of water from evaporation, (b) self-dessication during cement hydration, and (c) temperature changes. Others have noted that the salient reason is due to the loss of water during curing and that as a result the most important time occurs initially after construction. Theoretical studies have repeatedly demonstrated that shrinkage stresses on the surface far exceed known

tensile strengths and thus imply that cracking is inevitable for normal cement-treated materials.[106-107]

Ideally, a cement-treated layer should possess high tensile strength coupled with a low tensile modulus. Such a material is suggestive of a rubber or plastic polymer. In addition, bitumen has properties similar to this and Otte[108] and Bonnot have suggested its use along with cement to increase the relaxation properties of the material. A great many types of **additives** have been suggested and studied for control of shrinkage cracking. However, current research has been primarily confined to laboratory studies and the need for controlled field tests has been noted.[106]

Wang[106] has presented an excellent summary concerning the use of additives for this purpose. Recalling the primary mechanism (surface water loss) of shrinkage cracking as well as the interaction of strength to imposed shrinkage stresses, the additives may be grouped into several categories. They are:

a. Hydroscopic (sodium calcium chlorides and/or sugar that primarily reduce the moisture loss and thus decrease shrinkage).

b. Water-reducing additives (lignosulfates that primarily reduce the optimum moisture content for field compaction while increasing the dry density).

c. Flocculating agents (lime for high clay content soils (this additive normally alters the plasticity characteristics of the soil, i.e. reduction of adsorbed water on clay surface)).

d. Reduction of hydration heat (fly ash, whose chief function is the low heat of hydration along with a slower gain in strength, thus decreasing the amount of volumetric contraction due to thermal effects). Along these lines, Zube, et al.[109] has shown, based upon an evaluation of existing cement-treated pavements, that type II cement is better than type I cement.

e. Surface sealing and hardening (sodium silicates and hydroxides that improve the loss of moisture at the surface while increasing the tensile strength).

f. Expansive additives (primarily a direct result of an expansive reaction between the sulfates and aluminates in the cement. If the layer is restrained, the expansion results in a "prestress" effect due to the buildup of compressive stresses. Thus the shrinkage stresses that do develop must first overcome this "prestress level" before mobilizing any of the tensile strength inherent in the treated layer).

Because many of these additives also may result in decreased strength, it is imperative that further studies be focused upon the field performance of these treated layers.

Norling has presented a summary of laboratory and field results concerning the minimization of reflective cracking in soil-cement pavements.[110] Field studies show that a wide variety of practices are followed and being tried for this distress. They include use of surface treatments, delayed asphaltic surface placement, use of higher penetration asphalt cements, delayed and/or stage construction, inverted or "upside-down" layer designs, and use of rubberized asphalts. In general, none of these methods have been found to completely eliminate cracking.

George[107,111] has conducted considerable theoretical studies leading to the development of a model for shrinkage cracking in soil-cement bases subjected to one-dimensional drying from the top face of the slab. The basic assumption of this theory is predicated upon cracking being advanced at the surface from preexisting surface flaws. It was shown that these preexistent flaws in a cement-treated layer can be easily formed in the compaction (rolling) process. The flaw theory used followed that first recognized by Weibull and concerns the localized strength of the material near the flaw. Continuum theory was used to predict the distribution of shrinkage stresses with depth in the layer. The effect of subgrade shrinkage with a plain strain finite-element displacement model was also used to investigate the manner in which reflective cracks would occur in pavements.

Pretorious and Monismith[112,113] have also developed a treatment for shrinkage stresses in pavements containing soil-cement bases. The procedure uses an incremental axisymmetric finite-element solution provided the creep characteristics of all material layers are known along with the shrinkage and strength properties of the cement-treated layer. The procedure is admitted to be an approximate solution to the complex problem. In essence, the solution is based upon a prior knowledge of the shrinkage strain distribution. If this is known or assumed, the shrinkage strains can be incremented in small time intervals, then allowing creep relaxation to occur within the time interval until maximum

shrinkage has been achieved or cracking occurs. The model will also estimate crack spacing as well as crack width.

THEORETICAL MODELS, LOAD-INDUCED STRESSES

From the previous paragraphs it can be surmised that there are several unique characteristics of cement-treated materials that directly concern the selection of a theoretical stress-strain model used in any load-induced fracture analysis. These features are the relatively high initial stiffnesses that are obtained with laboratory results. Such stiffnesses (perhaps in excess of 10^6 psi) leave the question open as to whether elastic slab or layered theory is applicable to the analysis. Furthermore, because shrinkage cracks are nearly inevitable, the applicability of a layered theory, interior versus edge load slab analysis, or the use of a finite-element program must be ascertained. In addition, the stress dependency (nonlinearity), repetitive load effect, differences in compressive and flexural tests and the question as to the direct applicability of a laboratory obtained stiffness value to be used in a model assuming horizontal continuity obviously make the question of modulus selection a difficult task in the analysis framework.

There appears to be no clear-cut consensus as to how to handle the stiffness of the treated layer. Yamanouchi[114] has reported that the stiffness of the cement-treated layer after N repetitions is of the form:

$$E_N = E_o(a + b \log N)$$

where E_o is the initial laboratory modulus, a and b are regression constants, and N is the number of repetitions. In this equation the E_N value decreases from E_o in semilog fashion. Lister and Jones[103] have used nondestructive vibratory tests on pavement sections having cement-treated layers at the Alconbury Test Road in England. They concluded that the stiffness of the cement-treated layer gradually approaches (with traffic applications) the modulus of an untreated

granular material. This result has prompted Pell and Brown to suggest the use of stiffness for cement-treated materials being equivalent to that of granular materials for design and analysis. However, Otte[108] has challenged Pell and Brown's concept by claiming that the initial laboratory modulus can be used, with reasonable engineering accuracy, within an elastic analysis framework. This concept also appears to be demonstrated from stress-strain-deformation comparisons of test sections analyzed at the University of California under the auspices of Mitchell and Monismith.[30,112,115-118] The results of these tests will be brought out in subsequent discussions of the validity of various theoretical models.

The applicability of a specific theoretical model for cement-treated material involves the basic questions of slab versus layered concepts, continuity of the layer versus discontinuity (cracks), and the ability to handle the nonlinear response of materials in the system. All of the reported work on cement-treated materials conducted by the PCA has involved use of Westergaard's slab theory.[105,119-122] In contrast, Otte[108] has investigated the applicability of linear elastic layered theory and concluded that it can be used to satisfactorily predict performance. Thompson et al.[123] used a linear elastic finite-element approach to analyze results of the Alconbury test sections. While these studies have been aimed primarily at performance prediction, the most extensive series of research studies regarding predicted and observed stresses-strains and deformations have been conducted at Berkeley (University of California).

Mitchell and Shen[115] used elastic layered theory to provide general insight into the thickness designs of soil-cement pavements using the results of laboratory failure stresses and strains. It was noted in this 1967 study that the use of layered theory was only an approximation, due to the potential cracking and weathering (curing) factors associated with the treated soil.

Later studies by Pretorious,[112] Fossberg,[116] Wang and Mitchell[117] and Fossberg, Mitchell, and Monismith investigated the applicability of three different theoretical models. They were a 5-layer linear elastic

system, an axisymmetric finite-element program, and a prismatic space finite-element program. Test pavements were constructed and the following parameters measured under load: (a) vertical surface deflection, (b) vertical stress at the top of the subgrade, (c) radial strain at the bottom of the stabilized (cement-treated) layer, and (d) vertical deflection near the bottom of the stabilized layer.

In general, the advantages of the layered system model are its relative simplicity and understanding; the finite-element (axisymmetric or central load) has advantages of incorporating nonlinear behavior for all layers, while the use of the prismatic space finite-element program has the added advantages of being able to investigate central loading and edge loading effects along with the analysis of "cracked" layers within the pavement system.

Keeping in mind that initial laboratory moduli were used as input, the following conclusions were obtained. In general, it was concluded that all methods were able to predict stresses, strains, and deformations reasonably well. However, both the elastic layered model and axisymmetric finite-element approach tended to slightly underestimate vertical deflections. Hence the use of a deflection criterion for performance may be subject to this shortcoming. Comparisons between observed and predicted vertical stresses were found to be satisfactory. While the same could be stated for the horizontal strains, the finite-element programs were concluded to be slightly better over the layered system for strains at the bottom of the surfacing as well as the cement-treated base. It was also observed that a load placed greater than 2 ft from an edge (crack) could be considered to have the same responses (stresses, etc.) as a centrally loaded (axisymmetric) case. However, as would be expected, edge loads produced the most severe cases of stresses and deformations. Pretorious and Monismith[30] subsequently used the prismatic space finite-element program to predict the typical type of crack pattern in a pavement having typical transverse shrinkage cracking. It was concluded that the predicted crack pattern would be very similar (i.e. ladder-type cracking) to crack patterns commonly found in actual pavements.

FATIGUE-DESIGN ANALYSIS

It has already been noted that the use of the horizontal tensile stress (strain) at the bottom of the cement-treated layer has been recognized by most current research as the major indicator of fatigue (load induced) fracture. As such, recent studies have focused upon the fatigue characterization of cement-treated materials. Initial studies dealing with this property were conducted by the PCA in the 1960's.[105,119-122]

The original PCA results by Nussbaum and Larsen[121] used a limiting deflection approach to soil-cement pavement design. Such an approach, although not incorporating the effects of load repetition directly into the required thickness function, did allow the evaluation of soil-cement laboratory properties. The design equation was:

$$\frac{wk}{p} = \alpha\left(\frac{a}{h}\right)^{\beta}$$

where w is the deflection ($w = 0.03$ in. used), k is the modulus of subgrade reaction, p is the applied pressure, a is the radius of the applied load, and h is the required soil-cement thickness. In the study, it was found that all of the soils stabilized could be grouped into two categories (i.e. fine-grained and granular) of equal response. Thus the α and β factors are dependent upon material type with $\alpha = 0.058$ and $\beta = 1.52$ for soil cement and $\alpha = 0.163$ and $\beta = 0.652$ for granular bases.

Further studies on soil cement were conducted by Larsen and Nussbaum[105] relative to the fatigue (repetitive load) effect. This and the subsequent study were consolidated to formulate a design thickness procedure using fatigue concepts by Larsen, Nussbaum, and Colley.[122]

In the fatigue studied conducted by Larsen and Nussbaum,[105] the derived relationship for three different soil cements were of the form:

$$\frac{R_c}{R} = aN^{-b}$$

where R is the radius of curvature for a given load and number of

repetitions, R_c is the critical radius of curvature obtained from static flexural tests (failure condition under one load), N is the number of repetitions to failure, and a and b are fatigue constants. In the study, it was found that a was a function of the beam thickness by:

$$a = 1.05 - 0.042h$$

In addition, the exponent b was found to be dependent upon the type of soil. The general relationships formulated for each AASHTO soil group investigated are shown below.

Soil Group	Fatigue Equation
A-1-b	$R = \dfrac{R_c \times N^{0.032}}{1.05 - 0.042h}$
A-2-4	$R = \dfrac{R_c \times N^{0.025}}{1.05 - 0.042h}$
A-4(3)	$R = \dfrac{R_c \times N^{0.054}}{1.05 - 0.042h}$

Curvature R values were determined for each test in two ways and subsequent studies showed a good agreement between both methods. One method used flexure theory and was

$$R = \frac{EI}{M} = \frac{h}{2\varepsilon}$$

with h being the beam depth and ε being the fiber strain.

The other procedure used a geometrical approach using curve fitting techniques to measured deflections along the beam. In this approach,

$$R = \frac{1}{\left(\dfrac{d^2w}{dx^2}\right)}$$

with $\frac{d^2w}{dx^2} = \frac{1}{12c}(-1w_0 + 16w_1 - 30w_2 + 16w_3 - 1w_4)$

with c being the distance between equally spaced points and w_i being the deflection at the i^{th} location.

Combining the results provided by both studies, the PCA developed a thickness design procedure using Westergaard's theory in the form of a fatigue analysis.[119] The allowable traffic repetitions for soil-cement pavements developed are:

$$N = \left[\frac{(1.77k)^{A_1}}{\frac{c}{f(h)}}\right]^{A_2} \times \left(\frac{\sqrt{a}}{P}\right)^{A_2}$$

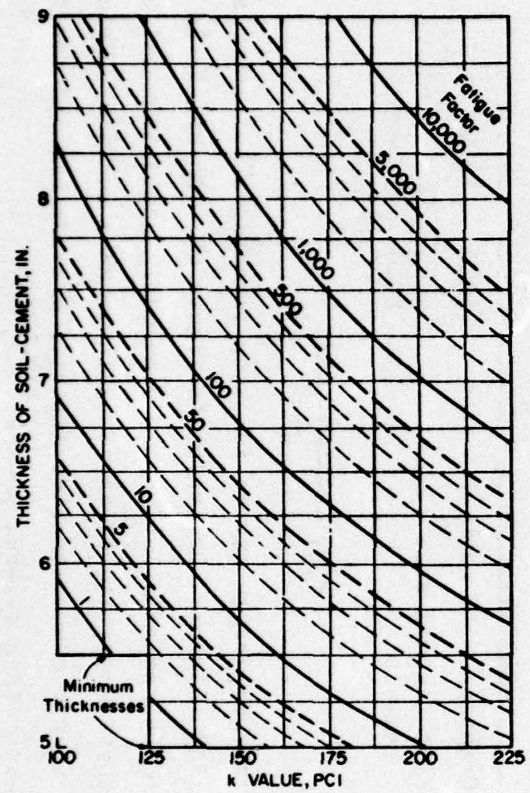
where k is the modulus of subgrade reaction, a is the radius of the contact area (inches), P is the wheel load (kips), A_1, A_2, and c are regression constants dependent upon cement-treated material type as shown below, and f(h) is a function also noted below (h in inches).

Material	Regression Constants A_1	A_2	C
Granular soil cements	0.3	40.0	10.4
Fine-grained soil cements	0.315	20.0	10.0

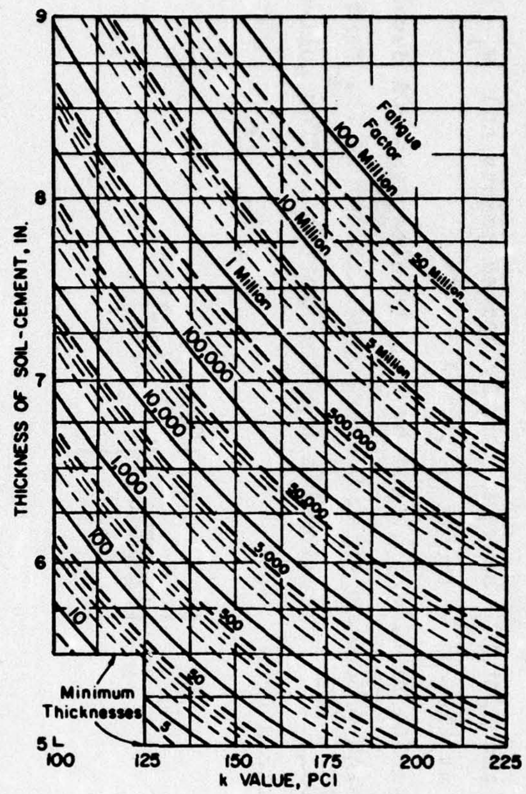
$$f(h) = \frac{(2.1h - 1)^2}{h^{1.5}}$$

Typical fatigue-design curves for both cement-treated materials for a 9K wheel load (highway) are shown in Figure 67. It should be pointed out that the design relationship previously noted does not consider any structural advantage of an asphalt concrete surface. Thickness reductions in the cement-treated material are recommended for thicknesses of AC greater than several inches.[119-120]

The other significant study concerning fatigue behavior of cement-treated material has been conducted by Pretorious and Monismith[30,112] at



Fine-grained soil cement



Granular soil cement

Figure 67. PCA soil-cement thickness design curves for highway fatigue loading (9-kip single wheel[104])

the University of California. In this study, fatigue tests (flexural) were conducted similarly to those previously described for asphalt concrete. Strain gages were used to determine actual recorded strains at the top and bottom of the specimens during repeated flexure. The results of the study are demonstrated in Figure 68. It is to be noted that the results are plotted in strain versus repetitions, exactly like typical results for asphaltic concrete fatigue studies. It can also be observed that the data are regressed in both semilog form (typical of pozzolanic fatigue results previously shown) as well as log-log form (typical of asphaltic fatigue results).

An extremely significant result of this study is also shown in Figure 68. The fatigue results were recalculated, in the same fashion as that proposed by Larsen and Nussbaum, using the parameter of radius

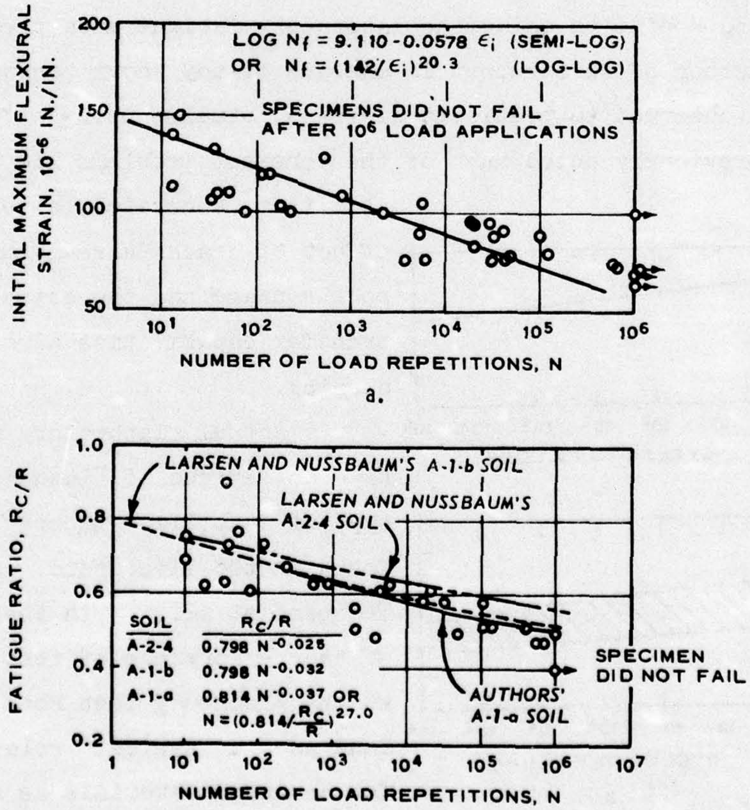


Figure 68. Soil-cement fatigue results (from Pretorious and Monismith[30])

of curvature in lieu of tensile strain. It can be seen that an extremely favorable comparison between the two procedures was obtained. In his discussion, Pretorious has noted that the results shown are for beam specimens having approximately the same "span" to "depth" ratio. Because the PCA results indicated an additional effect of specimen thickness, h , upon the fatigue results, Pretorious has questioned the validity by the known differences in behavior between "shallow" and "deep beam" action. The results of this study showed that the strain-repetition concept was as equally valid for cement-treated materials as the radius of curvature approach. Obviously, the use of strain (or stress) has more advantages in a theoretical model analysis than curvature values.

Although a theoretical pavement study has been conducted by Pretorious and Monismith using the laboratory fatigue data previously shown, the author of this report is unaware of any known performance correlation (observed to predicted distress) studies using a fatigue model. As previously noted much of the inherent problems lie in the ability to accurately assess the effect of cracks already present due to shrinkage and the actual load transfer capabilities across this opening.

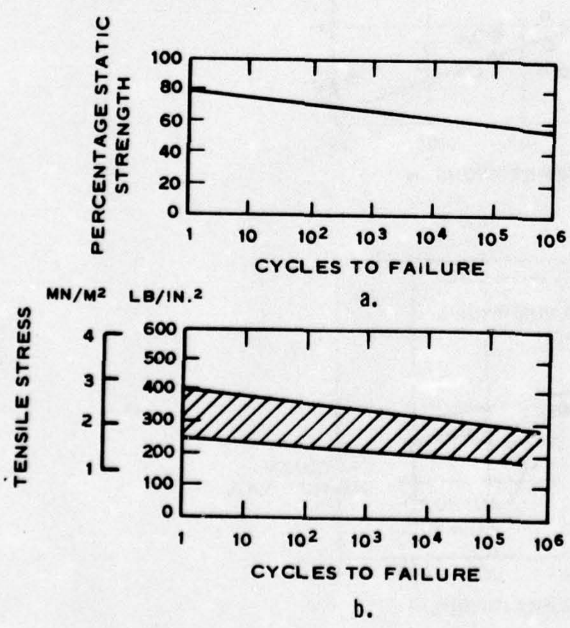


Figure 69. Typical fatigue relationship for lean-concrete mix (from Thompson et al.[123])

Actual laboratory data relative to fatigue of "lean-concrete" type of stabilized material were not found in the literature. However, Thompson et al.,[123] in their analysis of the performance of test sections at the Alconbury Test Road in England used a "typical" relationship for cemented materials as shown in Figure 69. Using the actual measured flexural strength ranges of the lean-concrete mix used in the

test, typical fatigue relationship ranges showing tensile stress versus log repetitions were used in the analysis.

PORTLAND CEMENT CONCRETE

BACKGROUND

The successful performance of PCC pavement is well documented in pavement technology. Because of its high stiffness (modulus) and ability to serve as a slab, the major design parameter for thickness design and performance is the flexural (tensile) stress induced by the traffic load. Because PCC, like all other rigid materials, is susceptible to repetitive load cracking from stress levels below that for ultimate static failure, much research work has been devoted to the laboratory and field performance of PCC pavements subjected to fatigue loading.

LABORATORY FATIGUE RESULTS

Although a great amount of work has been devoted to laboratory fatigue behavior of PCC, much fundamental research still needs to be evaluated. It is somewhat significant to note that many of the fatigue characteristics for PCC are similar to fatigue behavior of asphalt concrete.

Although many agencies (design) and technical papers endorse the existence of an endurance limit for PCC, Kesler[124] has stated that there is no evidence to support this concept for fatigue tests up to 10 million cycles. It is acknowledged, however, that a typical fatigue strength at 10 million repetitions is about 55 percent (applied stress to static strength ratio). Hence the commonly used endurance limit of 50 percent is perhaps justified in practice as designs (even in highway pavement analysis) for repetition levels beyond 10 million are generally no more than an academic exercise.

A typical laboratory fatigue curve for PCC is shown in Figure 70. It can be observed, that like previous fatigue curves for pozzolanic type materials, fatigue results are generally plotted in terms of the stress ratio (sometimes referred to as the fatigue strength) versus log of the cycles to fracture.

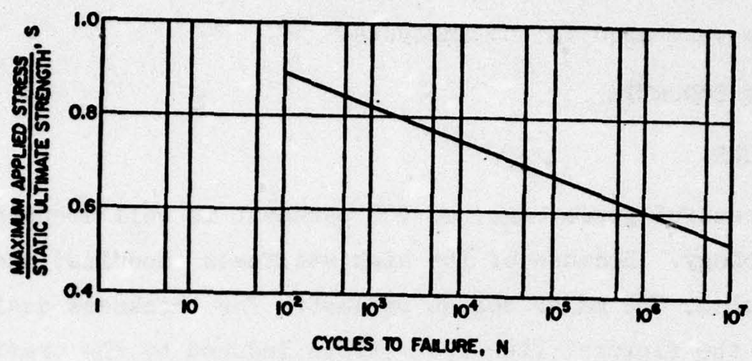


Figure 70. Typical PCC fatigue curve
(from Kesler[124])

It is significant to note that the relationship shown in Figure 70 is common to most types of PCC mixes. As a result, it can be concluded that many of the factors that affect static strength (e.g. water/cement ratio, cement content, curing, etc....) affect the fatigue strength in a similar proportional manner. Although some researchers have reported increases in fatigue strength, especially during the first 3 months, these studies have used a common age to evaluate the static strength of the material. Kesler[124] has suggested that if the static strength of the PCC were used at the same time (age of cure) that the fatigue tests were conducted, then the fatigue strength relationship would be independent of time.

Fatigue tests can of course be conducted in compression, tension, or flexure. Results appear to confirm that the fatigue behavior is dependent upon the type of stress as well as the stress gradient. As a result, fatigue tests are quite often plotted in terms of a Modified Goodman diagram.[112,124] In general, the fatigue life of a specimen subjected to a zero stress gradient (i.e. uniform stress state in tension or compression) shows the lower bound of fatigue resistance. Therefore, flexural tests conducted (a stress gradient greater than zero) will result in increased life.

Some research is also available to suggest that the transfer of uniaxial stress states, developed in fatigue testing, are not directly applicable for fatigue analysis of PCC subjected to a multiaxial (i.e.

triaxial) stress state normally induced by a load.[112]

For pavement purposes, the majority of fatigue testing is done in repeated flexurel modes of test. Exactly similar to the behavior of asphaltic concrete fatigue, the elastic modulus of PCC subjected to repeated loads generally decreases with load applications. Figure 71 shows such a typical response. In general, at about 70 to 80 percent of the failure repetitions, the rate of change in the modulus increases substantially. This reduction in moduli is directly responsible for the

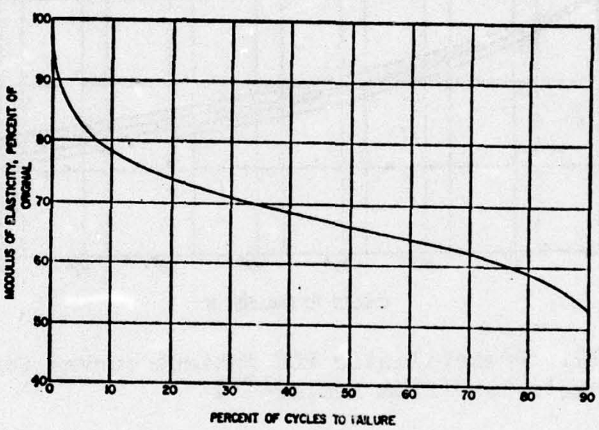


Figure 71. Reduction in modulus due to repeated load applications (from Kesler[124])

observed results that the tensile strain at failure, under repeated loads, is constant (about 250 μin./in. for all stress and repetition levels. In addition, this strain under repeated load failure is greater than that required for static results (about 190 μin./in.).

Laboratory studies have also shown that the load history (sequence of load applications) directly affects the fatigue strength. If a load near 90 percent of the ultimate strength is applied initially, there is evidence that the PCC sustains internal damage that cannot be recovered. However, if stress levels below about 50 to 55 percent are initially applied to the specimen, a beneficial effect is observed for both the static and fatigue strength of the material.[124] In addition, it has been found that the sequence of load (stress) level between these values alters the fatigue life of a specimen. This observation is in direct contrast to the validity for using Miner's linear damage law for

cumulative fatigue effects. Studies have indicated that application of this rule is unsafe for high loads but too conservative for low loads.[124] However, recognizing these factors and the relative ease to problem solutions using Miner's hypothesis, fatigue curves have been developed using a probabilistic concept to compensate for these effects. Such a relationship is illustrated in Figure 72.

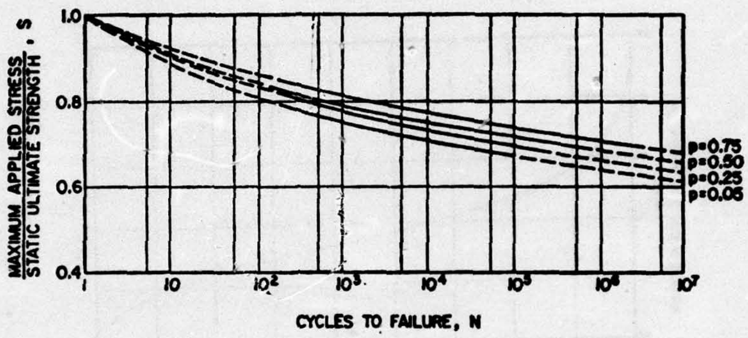


Figure 72. Probabilistic PCC fatigue curves for use with Miner's law (from Kesler[124])

Finally, fatigue tests have indicated that the speed of testing (frequency) results in negligible differences in fatigue behavior, at least between about 1 to 10 Hz. This is similar to reported fatigue results with asphaltic concrete. However, a very significant result deals with the effects of rest periods upon fatigue behavior. It has been found that periodic rest periods increase the fatigue life of the PCC specimen. The increase appears to be more pronounced for longer fatigue life and also increases for rest periods up to about 5 min. This is illustrated in Figure 73. It is interesting to compare this figure with that reported by Van Dijk for asphalt concrete (AC) fatigue (see Shell Oil Company section of this report). It appears that a plateau exists for both materials (PCC and AC) that defines the limiting beneficial effect of rest periods upon fatigue behavior.

From this discussion it should be apparent that one should not expect a precise solution from the direct application of laboratory fatigue results with a linear cumulative damage model approach for design. Fortunately, if the unconservative factors affecting fatigue

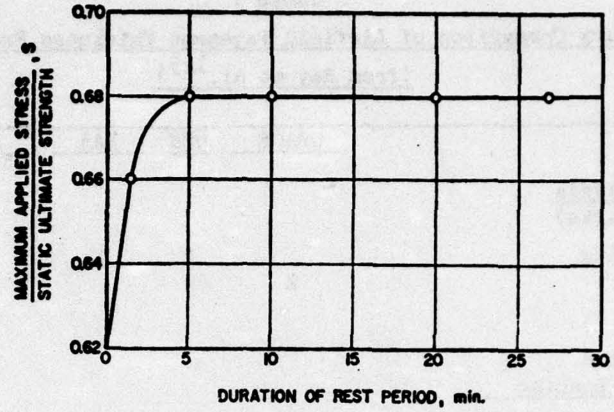


Figure 73. Effect of rest period upon PCC fatigue response (from Kesler[124])

behavior are recognized and accounted for (e.g. application of high stress ratios), use of constant amplitude fatigue testing with continuously applied loadings will generally result in a safe fatigue life estimation of the structure.

DESIGN APPLICATIONS AND FIELD PERFORMANCE STUDIES

General. Several design agencies have had successful design procedures for PCC pavements for many years. For airfield design in particular, the major agencies are the U. S. Army Corps of Engineers (USACE), U. S. Navy (USN), Federal Aviation Administration (FAA), and the PCA. While it is not the intent of this report to discuss the salient design features of each procedure, some of the major similarities as well as dissimilarities are noted between methods. Table 7 is a summary comparison of these design systems. It is interesting to observe from the table that there is not one major factor that all agencies mutually agree on. However, even though each agency uses a somewhat different "set of design inputs," Ray[125] has observed that all the procedures yield very similar results. This is a direct consequence of the fact that all of the methods have been modified to account for years of experience and performance studies on actual airfield pavements.

Table 7

Summary Comparison of Airfield Pavement Thickness Procedures

(from Ray et al.[125])

	USACE	USN	FAA	PCA	Notes
Westergaard Analysis (Influence charts)					
Interior loading		X	X	X	
Edge loading	X				Assumes a 25% load transfer
Modulus of Rupture (MR)--3rd Point Loading					
28-day flexural strength		X			
90-day flexural strength	X		X	X	
Safety Factors (SF)					
Working stress, $f = \frac{MR}{SF}$					
Critical areas (SF)	1.54	1.4	1.75 or 2.0*	1.7 to 2.0	*2.0 when annual crit. departures >6000
Noncritical areas (SF)	1.30	1.2	0.9T	1.5 to 1.7	T = thickness of critical area
Subbase					
Assumed k-value, pci, or k-value	X	X	300	X	See FAA method
determined by plate loading			X		
Use of Steel					
Reduction in slab thickness permitted for specified steel, %	Yes	Yes	No	No	
k-Value Determined by 30-in. Plate					
Assumed or			X		
MIL-STD-621, 104	X		X		Corrected bend. + satur.
or ASTM method		X		X	Corrected bend.
Aircraft Gear Spacing and Wheel Contact Area					
Actual	X	X		X	
Assumed			X		Conservative
Traffic					
Coverage levels					
Critical areas	25,000 coverages				
Noncritical areas	5,000 coverages				
or Safety Factor		X	X	X	

Of relevant importance to the discussion of repeated load effects upon PCC pavement are the effects of the use of Safety Factors (it should be recognized that the Safety Factor is the reciprocal of the fatigue strength), particular design pavement areas, and the treatment of traffic effects upon thickness. Because of the previously noted importance of field performance observations upon the design method, a detailed discussion of the USACE and PCA procedure is presented. In addition, the results of a study conducted by Vesić and Saxena[126] is briefly described for the repetitive load effect observed from the AASHTO Road Test for PCC highway performance.

U. S. Army Corps of Engineers Procedure. The U. S. Army Corps of Engineers design procedure for PCC pavements is, in the author's opinion, the most extensively developed and field proven design procedure currently existing. The method has been based upon a combination of theoretical studies, small-scale model results, full-scale accelerated test track studies, and numerous condition surveys of existing rigid airfield pavements.

In the overall design procedure, the effects of mixed traffic are ignored in the analysis. Normally, design is based upon a critical aircraft that generally is the heaviest anticipated (most damaging) in the design life. The repetitive fatigue effect of traffic applications is, however, accounted for directly by the use of a "Design Factor." (Design Factor (DF) is defined as the strength to stress ratio and hence is the reciprocal of the fatigue strength.)

It is very significant to note that the relationship between the DF and traffic volume (expressed in the USACE procedures by traffic coverages) has been derived mainly from field performance studies of accelerated test tracks and not from any laboratory fatigue work. The relationship used by the USACE is shown in Figure 74. The dashed line is only used for B-52 aircraft and was generated to account for dynamic load increments of approximately 15 percent due to observed porpoising of the aircraft during taxiway operations as well as observed pavement performance studies. Even though the relationship shown is from

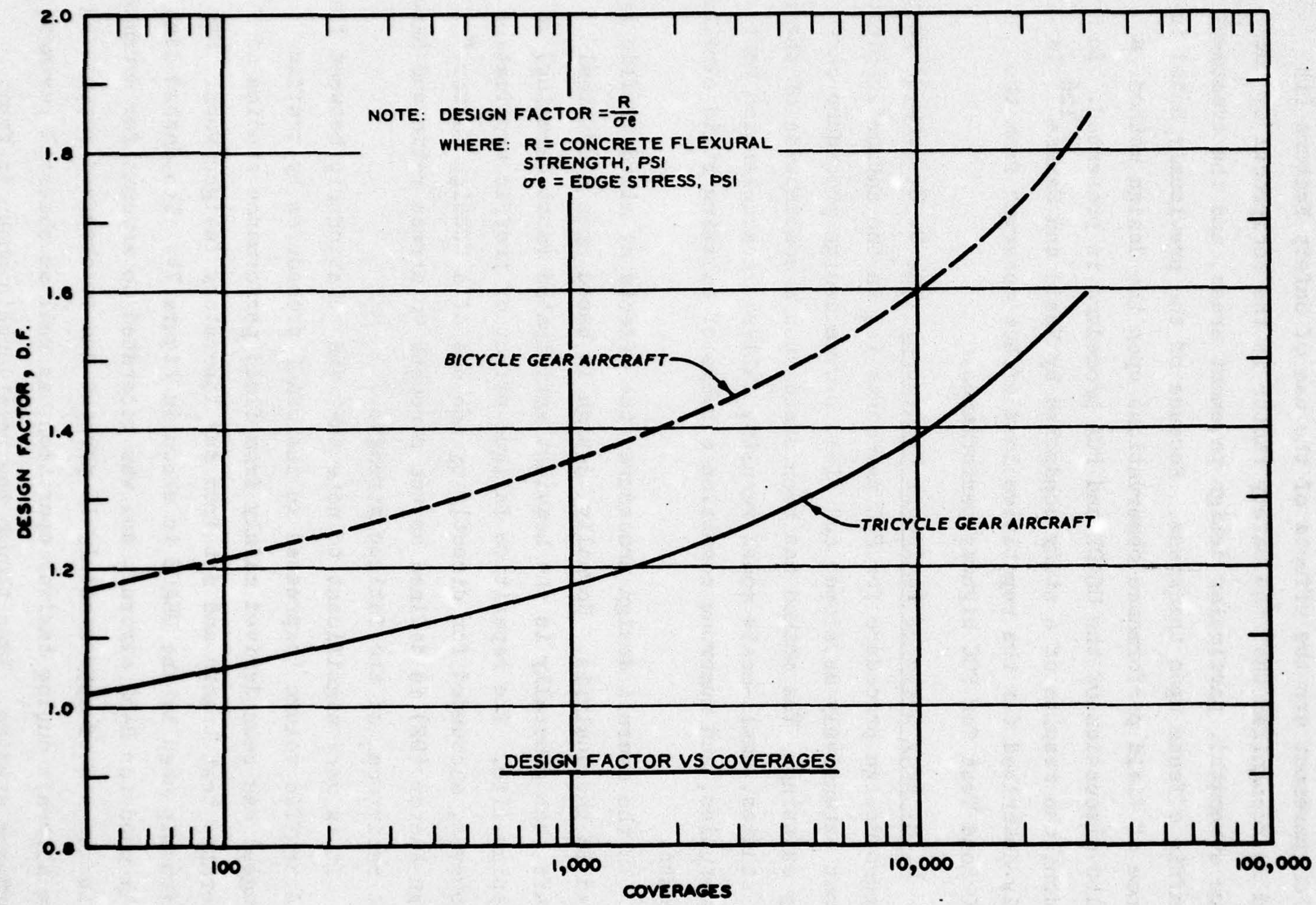


Figure 74. USACE field fatigue performance relation (from Hutchinson[127])

performance studies, an excellent comparison to the laboratory fatigue data generated from Figure 70 would be observed within the range of coverage levels shown in Figure 74.

The DF curve shown is considered applicable to an "initial crack" condition. The USACE uses this criterion as a "failure condition."[127,128] Relative to the relationship of distress to performance, the USACE uses three groupings to identify various levels of PCC distress associated with cracking. For a pavement system (in contrast to a test slab), "initial failure" is defined when 50 percent of the slabs in the travelled way first crack (i.e. 2 or 3 pieces). A "shattered slab" condition occurs when half of the slabs become broken into 6 pieces and 30 percent break into 2 to 3 pieces. A "complete failure" condition is defined when 50 percent of the slabs are broken into approximately 35 pieces or more with the remaining slabs in lesser degrees of deterioration.[127,128]

From a comparative analysis of the USACE procedure to the results of the AASHTO Road Test, Rice[128] has developed a general relationship between the serviceability index (PSI) and failure levels defined by the USACE. It was found that a psi of 3.0 to 3.3 related to the "initial failure" condition; a PSI of 1.1 to 1.6 was equivalent to the "shattered slab" condition; while the "complete failure" category was beyond any PSI level observed at the AASHTO facilities. Figure 75 compares the results of the AASHTO Road Test by actual coverage to failure versus the design factor. Also shown is an extension of the DF versus coverage relationship used by the USACE. It can be seen that a conservative solution is indicated for the USACE relationship.

Another significant finding from USACE test track studies concerned the effect of the subgrade support conditions upon crack propagation. It was observed that for subgrades having k values less than 300 pci, the number of load applications between an "initial failure" condition and "complete failure" was less than on subgrades having a k in excess of this value. Hence it would appear that an inverse relationship between the crack rate (dC/dN) and the subgrade support (k value or strength) exists from these studies. Based upon

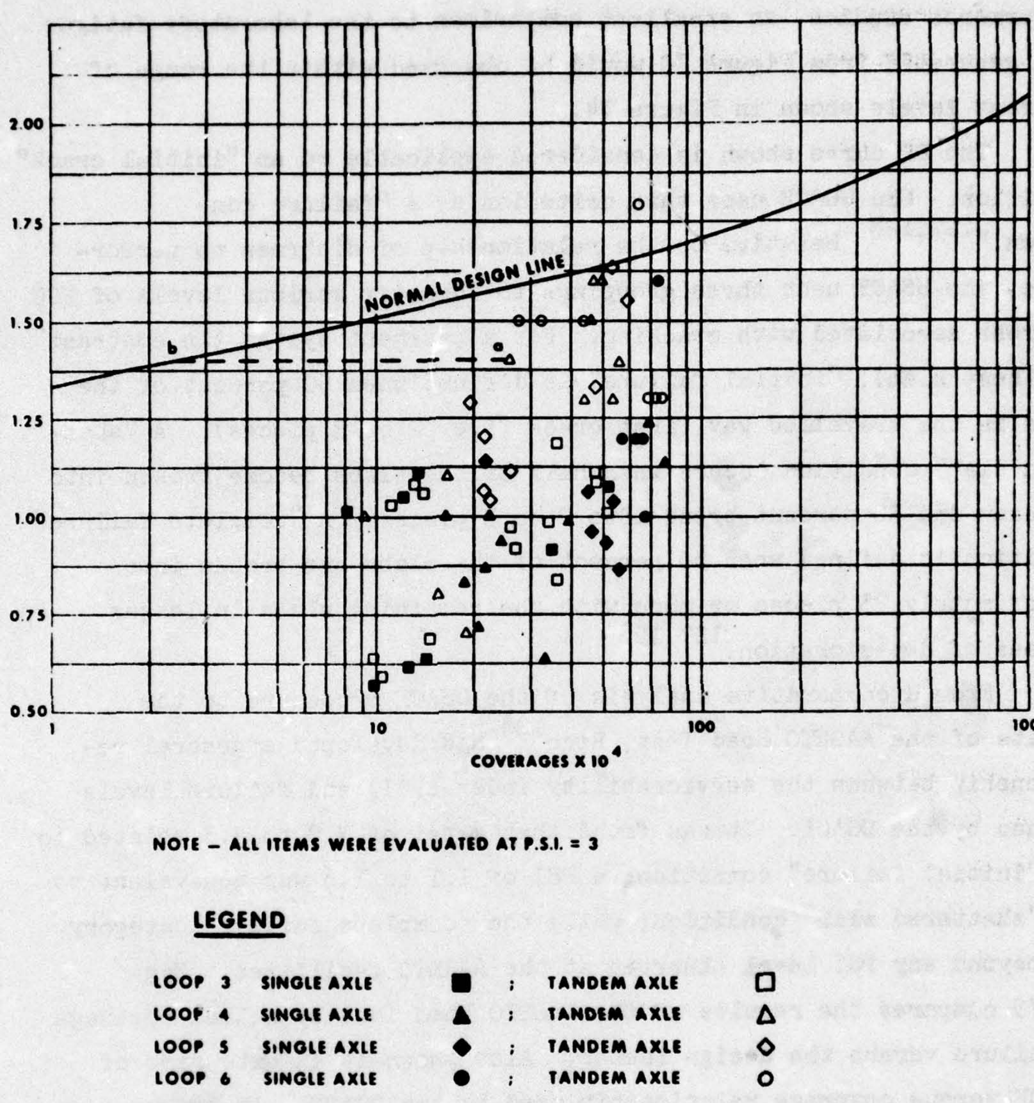


Figure 75. Comparison of USACE design factor-coverage relationship to AASHTO Road Test results (from Rice[128])

these results, thickness reductions for stronger subgrades were inherently built into the design procedure.[127]

A similar type of logic was also applied to performance results on reinforced concrete pavements. It was observed that even though the reinforcing did not increase the resistance to fracture, it did functionally serve to keep the cracks intact and delay distress due to

repeated applications of both traffic and environmental stresses. Hence it was found that reinforcing would result in an even longer "life" and, as a result, thickness reductions were developed from performance studies to account for increases in steel percentage. The effect of steel reinforcement upon thickness used by the USACE is shown in Figure 76. It should be noted that reference to Table 7 indicates that the FAA and PCA procedures do not allow a reduction in slab thickness with reinforcing steel percentage.

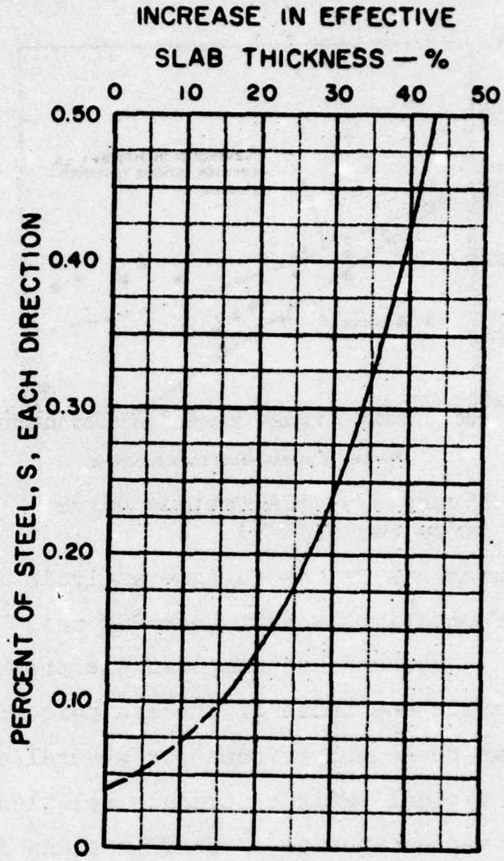


Figure 76. USACE criterion for PCC thickness reduction due to reinforcing (from Hutchinson[127])

PCA PROCEDURE

The PCA design procedure for airfield pavements accounts for the effect of repeated loads upon thickness by one of two ways. The oldest and probably most commonly used method involves the selection of appropriate safety factors that are applied to the flexural strength. The actual magnitude of these values depends upon the specific pavement area being designed (i.e. runway interior, taxiway, etc.) and hence reflects, indirectly, the relative damaging effects of both aircraft and traffic level. Normally, aircraft mix effects are ignored and a critical (most damaging) aircraft is selected for design.

However, the most recent PCA airfield design manual,[129] has an alternate analysis by which cumulative fatigue analysis may be used with the entire mix of aircraft anticipated at the airport. The concept uses

a typical fatigue curve for PCC pavements as shown in Figure 77.[130] In essence, each aircraft is analyzed for its contribution to the predicted damage it will cause in the life of the pavement. A linear cumulative damage analysis (Miner's hypothesis) is used to ascertain if the estimated pavement thickness will have a damage less than one. An endurance limit of 50 percent is assumed to exist in the analysis. Based, in part, upon USACE performance data, adjustments in the fatigue analysis must be made for pavements designed on foundations weaker than 200 pci.[129]

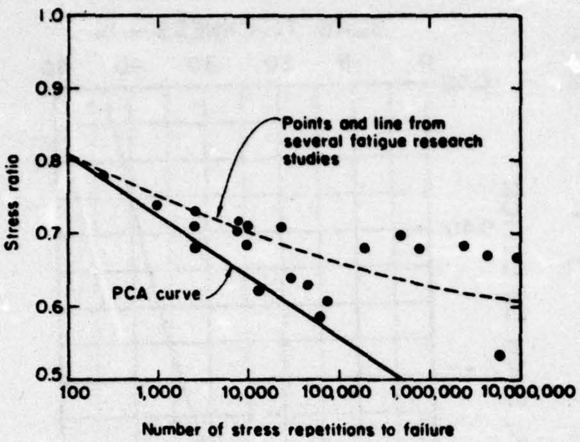


Figure 77. PCA fatigue curve (from Packard[130])

Packard has compared the theoretical results of the PCA fatigue analysis and those of the analysis proposed by the USACE.[130] Figure 78 shows these comparisons for several aircraft. The solid line indicates the typical fatigue-strength relationship of the PCA. The dashed lines are representative of USACE designs for various thicknesses and subgrade support conditions. The hatched area for the USACE analysis is indicative of the previously discussed thickness modification to account for added serviceability due to high strength foundations. For very low k values it can also be seen that the PCA relationship is somewhat unconservative. This is accounted for in the PCA design by the previously stated adjustment for weak subgrade support that must be included in the fatigue study.

AASHTO ROAD TEST,
Vesić and Saxena

Vesić and Saxena[126] have recently analyzed the rigid pavement results of the AASHTO Road Test. The structural behavioral study indicated that the subgrade response was comparable to an ideal isotropic elastic-solid. However, mathematical models were developed to arrive

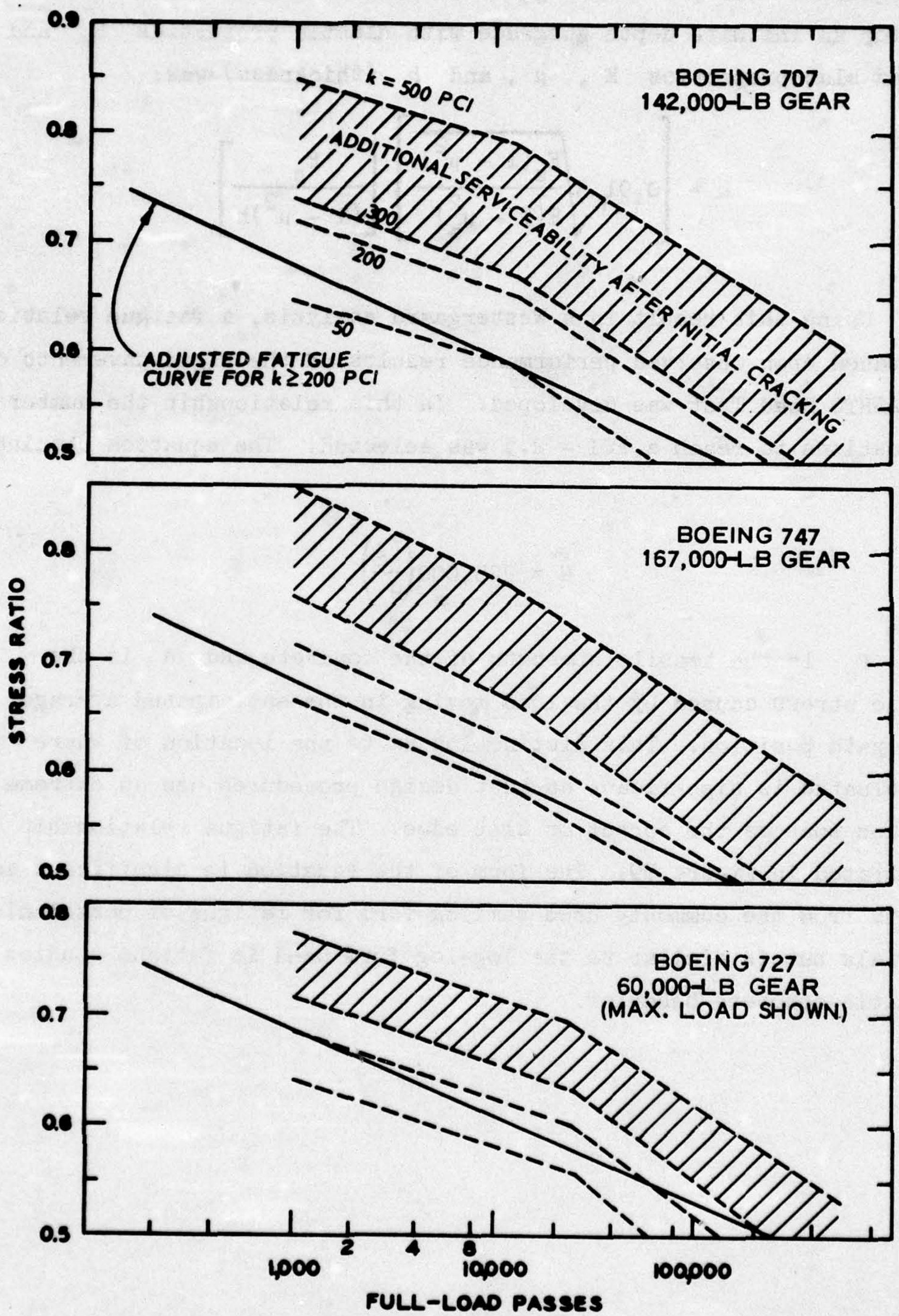


Figure 78. Comparison of PCA and USACE fatigue strength relationships (from Packard)[130]

at a comparable "Winkler" subgrade modulus that could be used with Westergaard's theory of rigid pavement stress analysis. The relationship for an infinite depth subgrade with elastic properties E_s and μ_s and slab properties E, μ, and h (thickness) was:

$$k = \left[0.91 \sqrt[3]{\frac{E_s(1-\mu^2)}{E(1-\mu_s^2)}}\right]\left[\frac{E_s}{(1-\mu^2)h}\right]$$

Using this result in a Westergaard analysis, a fatigue relationship based upon observed performance results of the rigid pavements of the AASHTO Road Test was developed. In this relationship the number of applications to reach a PSI = 2.5 was selected. The equation obtained was:

$$N = 225{,}000\left(\frac{f_c}{\sigma}\right)^4$$

where f_c is the tensile strength of the concrete and σ is the tensile stress caused by the load moving in the anticipated average wheel path position. This distinction as to the location of where σ is evaluated is significant as most design procedures use an extreme position such as the corner or slab edge. The fatigue relationship is illustrated in Figure 79. The form of the equation is significant as it departs from the commonly used semilog form for fatigue of pozzolanic materials but is similar to the log-log form used in fatigue studies for asphaltic-concrete behavior.

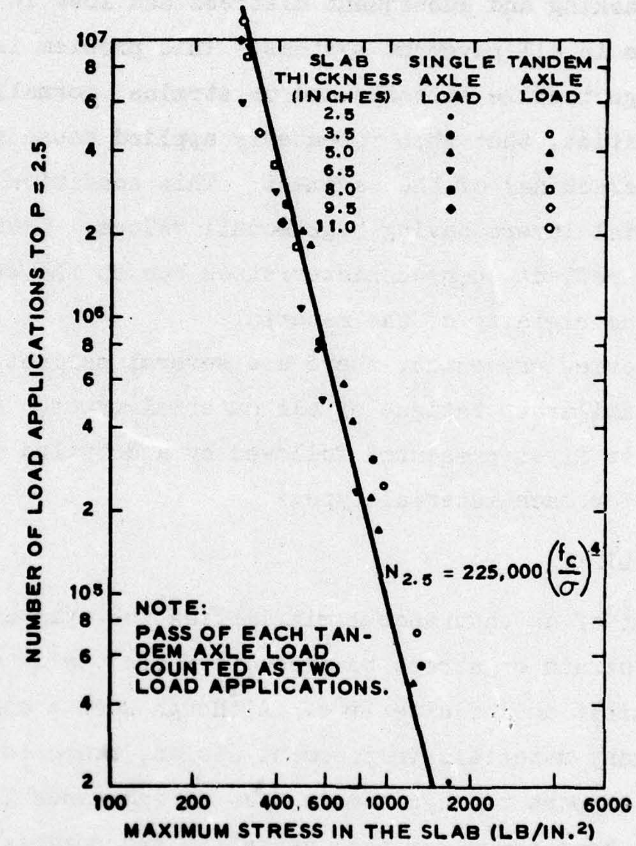


Figure 79. Fatigue relationship determined from the AASHTO Road Test (from Vesić and Saxena 126)

DISCUSSION OF RESULTS

GENERAL RESULTS

The previous chapter has described in much detail prior research and design activity related to fatigue cracking. In general, it can be concluded that cracking and subsequent distress and loss in performance is a major problem in all pavement systems. This problem is due to the development of high tensile stresses and/or strains, normally less than the ultimate condition, that when repeatedly applied cause an eventual fatigue failure (cracking) of the pavement. This condition is much more probable in material layers having high moduli values. Thus, many stabilized materials reflect such characteristics due to the stabilizer, which increases the rigidity of the material.

From the review presented, there are several salient characteristics that are similar to fatigue of all material types. A discussion of these factors is first presented followed by a detailed discussion of the major points for each material type.

ENDURANCE LIMIT

The concept of an endurance limit implies the existence of a level of stress (strain or stress to strength ratio) below which the specimen will exhibit an infinite life. Although such a concept has been applied to many materials in pavement design, there is no laboratory evidence to support the hypothesis that an endurance limit actually exists. This has been shown for both asphaltic and pozzolanic (PCC) materials at least up to 10^7 load cycles.

However, even though this is an observed fact, some engineers have widely adopted the value of about 50 to 52 percent (stress to strength ratio) as being equal to the endurance limit for PCC and other pozzolanic materials. For asphaltic concrete fatigue, Monismith has suggested a tensile strain of 70 μin./in. (regardless of mix stiffness) as being a limiting endurance value. (See Figure 19 on page 40.) However, no other researchers have noted the existence of such a value.

Finally, in many respects, the agreement as to whether or not an

endurance limit does exist, is probably more of an academic than practical question. For postulated endurance limits previously noted (i.e.,...50 percent stress to strength value and 70 μin./in.), the number of repetitions to failure (fatigue strength) is probably in the range of 10^7 cycles. Except in a few extraordinary design situations would there be a real need to consider load applications of this magnitude or greater in design.

STOCHASTIC NATURE OF FATIGUE

A review of the fatigue relationships should immediately make the reviewer aware of the great amount of scatter or variability associated with fatigue tests on any material. (See, for example, Figures 4, 5, 15, 41, 61, 63, 68, and 77.) Although it is not the intent of this report to dwell on the use of statistical treatments of fatigue and other distress modes, the great variability obtained is an extremely important design consideration. In addition, it should also be noted that not one of the recommended design subsystems for fatigue currently incorporates the stochastic nature of these results.

For asphaltic concrete fatigue, research has illustrated that the distribution of fatigue lines at a particular stress level can be represented by a normal distribution if a logarithmic transformation of the life is used. This has been substantiated by both Pell and Monismith.[4,8,20] Thus the frequency distribution of fatigue (fracture) life is denoted by:

$$f(y) = \frac{1}{\sigma\sqrt{2\pi}} \times e^{-(y-m)^2/2\sigma^2} .$$

where

$f(y)$ = normal density of Y

$Y = \log N$ (transformed variable)

m = mean of Y

σ = variance of the distribution of Y

Naturally, the above distribution is considered to be applicable for a particular strain level. A comparison between predicted and

observed distributions obtained for 100 fatigue tests (one loading condition) is shown in Figure 80. Work by Monismith has also suggested

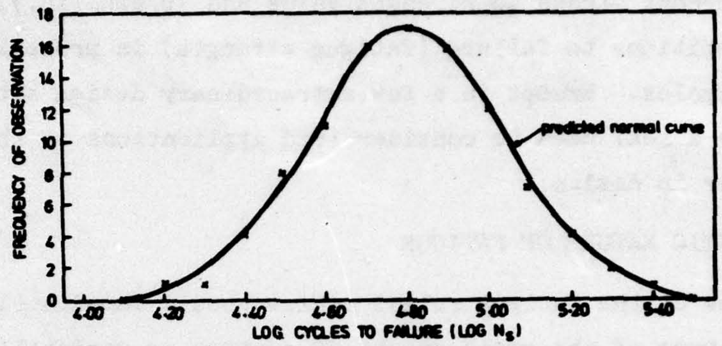


Figure 80. Predicted and observed frequency distribution fatigue patterns (from Pell[1])

that the variance of the fracture life is dependent upon stress level. Such a relationship is shown in Figure 81.

Figure 70, previously shown, indicated a typical fatigue relationship for PCC material. However, this material, like most others, exhibits great variability. As a result, the fatigue relationships shown may really be thought of as the 50th percentile fatigue curves. Taking into account statistical variations at other confidence levels, other fatigue curves may be defined. Figure 82 illustrates a typical set of fatigue curves for probabilities of failure for PCC material.[124] As can be seen, a very great difference occurs between the fatigue curves for different probability levels.

In summary, it can be observed that the statistical variation associated with fatigue tests is very significant for all pavement materials. In addition when one also considers the added variability associated with field construction (in situ variability), the combined effect will be of great magnitude. As there is presently no accepted design procedure for fatigue that adequately encompasses the stochastic nature of this distress, it would appear that failure to consider fracture from a probabilistic viewpoint is one of the major faults in present day design technology.

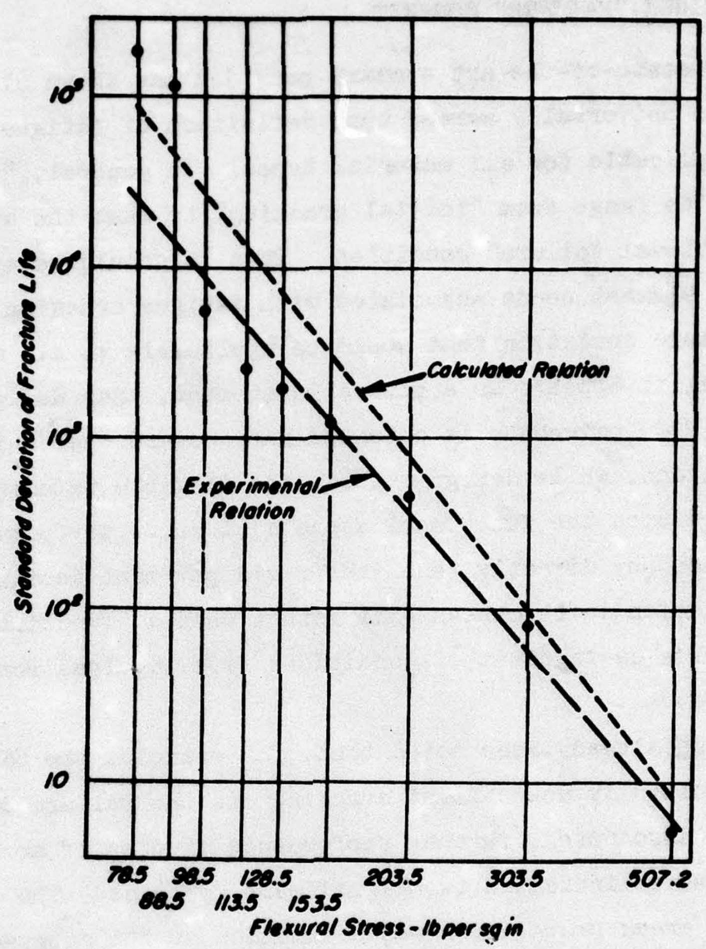


Figure 81. Dependence of fracture life standard deviation upon stress level for asphalt concrete (from Monismith[16])

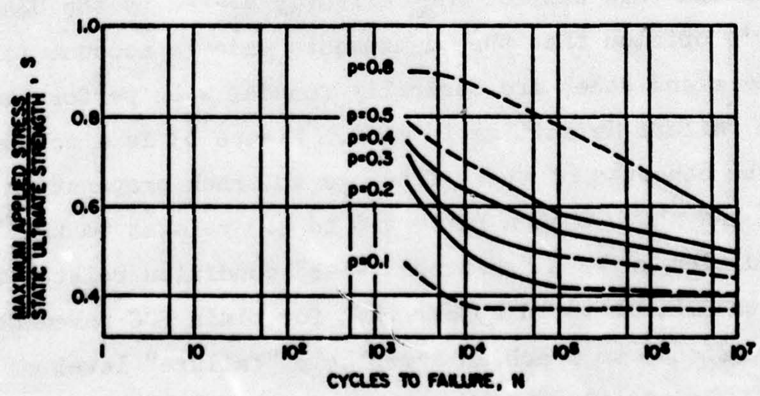


Figure 82. Probabilistic fatigue curves for PCC material (from Kesler[124])

FATIGUE "FAILURE" CONCEPT

The-state-of-the-art summary provided has shown clearly that there is no universally agreed upon definition of fatigue "failure" that is applicable for all material types. In general, "failure" has been shown to range from "initial cracking" to what the author terms as a "functional failure" condition. Thus it should be apparent that one of the biggest needs associated with fatigue cracking is to define such a failure condition that would be applicable to all material types. Furthermore, it appears as a general statement, that design methodology for rigid (PCC) pavements is oriented more toward the "initial cracking" definition, while design methods for flexible pavements have been focused more upon the functional aspects (i.e.,...performance). Such a design philosophy directly penalizes rigid pavement design; a proportional reduction in thickness will result when a "functional failure" is accepted for a certain set of conditions (i.e.,...load repetitions or service life).

It has already been noted that, for example, the USACE-PCC design procedure initially used "first cracking" as the failure determinant. However, as also noted, further performance studies of accelerated test pavement studies indicated two significant features. The first was that the rate of crack propagation was a function of the subgrade support and second, that the effect of steel (reinforcing) allowed a decrease in thickness to achieve the same performance level.

Although this has not been directly stated by the USACE, it is the author's opinion that the adjustments made to account for both subgrade support and steel are basically founded with performance oriented-functional failure definition in mind. Figure 83 is a schematic diagram relating the concepts of these findings to crack propagation rates. As Rice[128] has shown that a PSI = 3.0 to 3.3 relates to the "initial crack" condition while a "shattered slab" condition relates to a PSI = 1.1 to 1.6, it would appear that for plain PCC pavements, the time (load repetitions) to reach an acceptable "failure" level of 2.5 is not very large. In addition to these observations, it must also be noted

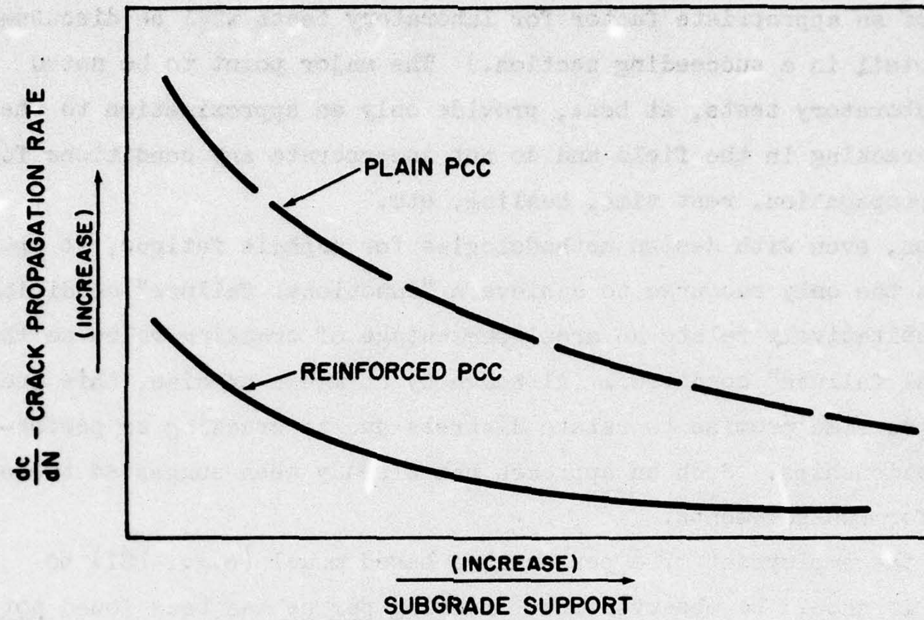


Figure 83. **Schematic illustration of effect of reinforcing and** ***subgrade support upon rigid pavement crack propagation rate***

that the use of a "functional" type of failure for fatigue of pavement systems must also be used as the final criterion if universal use is to be made for all pavement types. It must be obvious that the definition of "initial cracking" as a failure criterion cannot logically be applied to such pavements as continuously reinforced (CRC) pavements, where it is the design intent for regular short-spaced cracks to develop soon after construction. Finally, the use of a "functional failure" condition with the Vesić-Saxena equation for rigid pavements should be obvious.

A similar state of confusion also exists between the various asphaltic concrete fatigue criterion and design systems previously described. For example, both the Kingham and Kentucky (to a lesser extent) are fatigue cracking criteria developed from conditions of "functional failure" rather than "initial cracking" conditions. In contrast, most laboratory derived fatigue curves have the added problem of trying to use field factors to adjust laboratory to field conditions. As noted, Pell and Brown have suggested these factors to range between 5 and 100. Others, like the Shell organization, state the range to be from 3 to 10.

(The use of an appropriate factor for laboratory tests will be discussed in more detail in a succeeding section.) The major point to be noted is that laboratory tests, at best, provide only an approximation to the onset of cracking in the field and do not incorporate any conditions for vertical propagation, rest time, healing, etc.

Thus, even with design methodologies for asphalt fatigue, it appears that the only recourse to achieve a "functional failure" condition is to quantitatively relate an areal percentage of cracking value to the "functional failure" condition. Although by no means precise, this procedure holds some promise to relate distress due to cracking to performance relationships. Such an approach has already been suggested to be employed for PCC pavements.

In the employment of a performance based model (e.g., PSI) to distress, it should be observed that cracking <u>per se</u> has been found <u>not</u> to substantially lower the PSI value. Finn has demonstrated that using the flexible pavement PSI equation from the AASHTO Road Test, would change the PSI from 5.03 to 4.70 if the pavement were 100 percent cracked. Such statements, unless carefully interpreted, may give the false impression that cracking does not affect performance. Thus it appears that the major influence of cracking is that it significantly contributes to an increase in roughness and deformation which in turn significantly alters the PSI value. As a result, even if fatigue cracking is the primary or initial distress mode, it is the subsequent deformations and associated changes in profile (roughness) that occur as a result of cracking, that are the principal reasons for serviceability loss. In the author's opinion this is a very important concept to understand.

As a result, correlations of percent cracking to various levels of serviceability are an indirect attempt to correlate roughness, <u>induced primarily by load cracking</u>, to serviceability. It should be apparent that such a concept of fatigue failure may also be applicable to all material and pavement types (i.e. besides asphaltic concrete). Various investigators have proposed various levels of areal percent cracking to define various stages of functional condition. Zube[109] has suggested

procedures for asphalt concrete can be grouped into one of several categories. They are: (a) designs using limiting strain criteria, (b) phenomenological lab tests, and (c) mechanistic (fracture mechanics) approach. It appears that each of these approaches may be viewed in the order presented as representing various "generations" of fatigue approaches. The first generation or "limiting strain" criterion approach has been used for design since the early 1960's with the introduction of the Shell design method for highways. Since that time, other organizations (e.g., TAI, Kentucky Highway) have developed and adopted their own procedures. Nonetheless, even though criteria may slightly differ, the fundamental concepts of all of these techniques are quite similar.

Although all of these approaches may be considered as a first-order approximation for asphalt concrete fatigue design, they all suffer from the inability to incorporate the specific fatigue behavior of the precise mix that is to be used in the pavement. Various laboratory studies concerning fatigue behavior already presented have dramatically shown that variations in fatigue behavior between various mixes may be quite drastic.

Hence, the "second generation" procedures were aimed at conducting laboratory tests based upon phenomenological procedures that could supplement the "typical" or "provisional" fatigue response used in the first-generation design procedures. Several researchers, such as Pell, Verstraeten, and Kirk, have concentrated their research activities toward the development of a procedure that could be used to predict the typical laboratory fatigue response of any particular mix. Such an approach has the advantage of accounting for the exact mix characteristics to be used in the pavement while not having to physically conduct time-consuming and somewhat expensive fatigue tests on a routine basis. Others, such as Monismith and Chevron, have proposed a typical set of fatigue curves that can be adjusted for void properties of the mix.

The third-generation approach based upon mechanistic principles has been noted to afford great potential for fatigue analysis. However, it is the author's opinion that while such an approach has many potential advantages, it still must be currently viewed as a future type of

fatigue methodology. Thus, it appears that the best approach of today and the near future appears to be based upon phenomenological studies and design-oriented methods.

FUNDAMENTAL DIFFERENCES BETWEEN PROCEDURES

In this report several typical or provisional types of fatigue curves have been presented by various researchers or research agencies. Each, in itself, has been developed upon certain principles or concepts that follow engineering logic. However, there exist differences in the criteria that relate not only to thickness design differences but also to differing implications relative to the effect of temperature and, therefore, modulus and, subsequently, mix characteristics for fatigue behavior. In order to best illustrate these differences, a typical example for fatigue analysis has been developed for a 35-kip single-wheel load, tire pressure of 245 psi, full depth asphalt thickness of 12 in. resting on a subgrade of 30,000-psi modulus.

Figure 84 illustrates the fatigue results in terms of the number of repetitions to failure as a function of the asphalt concrete modulus E_1 for five basic criteria discussed in this report. They are the Pell and Brown criterion (Figure 8 using laboratory results directly as well as the field adjustment factor of 20), the typical Monismith criterion (Figure 19), the Kingham criterion (Figure 35), the typical Shell Oil criterion (Figure 37), and the Kentucky criterion (Figure 47). It should be observed that the reciprocal of the N_f value plotted in Figure 84 represents the unit damage. Thus the larger the N_f value, the smaller (less damaging) the unit damage is.

Table 8 represents a summary of a cumulative damage analysis for two extreme environmental conditions. Case I is typical of a very warm locale while Case II is representative of a very cold environment. The table shows the design number of repetitions that result from the analysis of each of the aforementioned criteria for each temperature condition. From the results shown in both Figure 84 and Table 8, the following general conclusions can be observed.

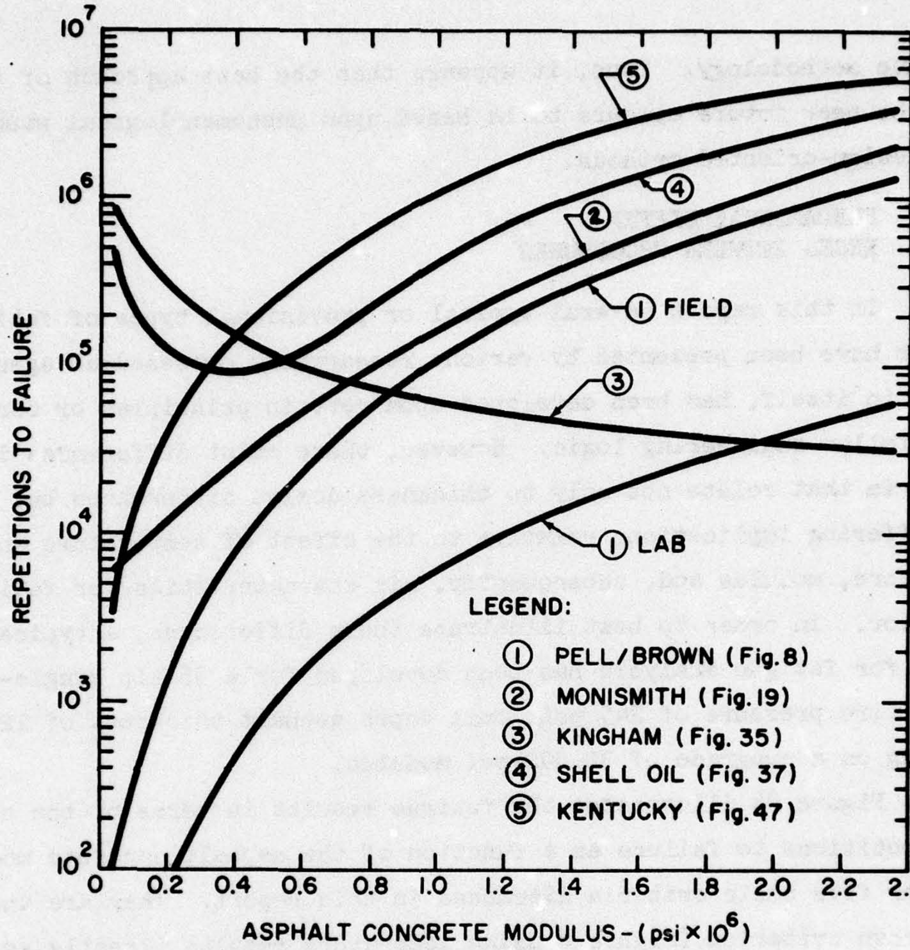


Figure 84. Comparison of predicted fatigue repetitions for different criteria

Table 8

Repetitions to Failure (N_f)

Criterion	Case I Warm	Case II Cold	$\frac{N_{f\ II}}{N_{f\ I}}$
Kentucky	52,080	714,290	13.7
Shell	128,200	243,980	1.9
Monismith	14,620	92,590	6.3
Kingham	222,220	72,460	.3
Pell (Field Adj)	1,570	52,630	33.5
Pell (Lab)	80	2,630	33.5

a. There is no universal agreement as to the relative damaging effect of temperature (AC stiffness) upon unit damage. At low stiffnesses (high temperature) both the Shell and Kingham criteria show less damage than at higher stiffnesses. In contrast, the criteria developed by Kentucky, Monismith, and Pell/Brown generally demonstrate that low temperatures are the most damaging.

b. The fatigue criterion developed by direct lab testing (Pell/Brown) is the most conservative for all cases investigated. It can be noted that even with an adjustment factor of 20 applied to the lab curve the unit damage is still one of the greatest indicated for almost all E_1 values.

c. The Pell/Brown curves show the greatest ratio of repetitions to failure changes between environmental locales (Case II to Case I ratio).

d. The Kingham criterion results in a thickness increase in cooler environments. This is in contrast to all other procedures which would indicate a less severe state of damage for this environment.

e. In general, there is not a very appreciable difference in design repetitions, at moderate climates, between the Monismith and Kingham criteria. This is also true to a lesser extent for the Shell criterion and the adjusted Pell/Brown curve.

Even though these observations are based upon only one example solution, the trends between criteria can be generally accepted for most design conditions. Thus, the use of the unique fatigue line (independent of AC stiffness) will always result in the most severe case (most conservative design solution) because of the strong influence of large tensile strains developed at low temperatures in the asphalt layer. In the opinion of the author there still remains a very significant question as to the proper influence of temperature upon not only the design thickness but also the proper interpretation of the effect of materials characterization (mix stiffness) upon fatigue behavior. Although the Kingham criterion is the only such criterion that significantly differs in concept between other criteria, it should be recalled that the other criteria have been based upon laboratory testing while the Kingham criterion was developed upon the field performance of numerous AASHTO Road Test pavement sections. Thus the question of a true relation between laboratory and field results may be quite significant in the interpretation.

In summary, even though there exists a general agreement in the overall fatigue design repetitions for moderate temperatures, differences do exist among the various criteria. In addition, the question as to whether warmer temperatures (low E_1, as interpreted from most laboratory tests), or colder temperatures (high E_1, as indicated from the field-derived Kingham criterion), are the most damaging still remains, in the authors viewpoint, unanswered.

VALIDITY OF PELL'S STRAIN CRITERION

In this report, it has been previously stated that Pell founded the concept that laboratory constant stress fatigue results could be plotted as a unique fatigue curve (in terms of log E versus log N_f) independently of test temperature and load frequency. This was accomplished through the use of the modulus (stiffness) of the mix, which is both time and temperature dependent. Careful review of the results of other researchers has indicated that this concept is not universally obtained from laboratory constant stress fatigue tests.

Previously noted fatigue research by Pell/Brown, Verstraeten, Kirk (flexural tests), and Bazin and Saunier has all verified that the "strain criterion" (i.e., unique fatigue relationship) does exist. In contrast, other researchers such as Monismith, Kirk (impulse), TAI, Shell Oil Company (original fatigue work), and Coffman have shown that different strain-fracture life curves are obtained for various temperatures. The latter research directly conflicts, of course, with Pell's and the other researchers' findings.

It should be recognized that each of the researchers discussed in this report has used a variety of test conditions (specimens, test equipment, load conditions, methods of determining strain) and so it becomes difficult to directly compare laboratory results. However, in the review process of the test conditions reported by all researchers, the author has found an interesting feature inherent in both research groups that either acknowledge or refute the concept of a "unique strain criterion."

Table 9 is a summary of researchers by grouping as to whether

they support the concept of the "strain criterion." In addition, the conditions of the applied repeated load are also indicated. From this table, it can be seen that each of the researchers reporting the existence (or verification) of a "unique strain criteria" used a continuously applied sinusoidal load. In contrast, other agencies found that a "unique strain criterion" did not exist and used (for the most part) some form of pulse load.

Table 9

Unique Strain Criterion Summary

Researchers/Organization	Load Conditions
Supporting Unique Strain Criterion	
Pell/Brown	Continuous-sinusoidal
Verstraeten	Continuous-sinusoidal
Kirk (flexural)	Continuous-sinusoidal
Bazin/Saunier	Continuous-sinusoidal
Supporting Multistiffness Criterion	
Monismith	Pulse-haversine
Kirk (impulse)	Impulse load
The Asphalt Institute (Lab)	Pulse-haversine
Coffman (Ohio State Univ)	Pulse-haversine
Shell Oil (original)	Continuous-sinusoidal*

* Unable to accurately determine directly from literature review. Potential (probable) load condition.

Even though it may be argued that such an examination does not constitute an absolute basis upon which to firmly establish a conclusion, it is the author's strong opinion that the concept of a "unique strain criterion" is a direct result of test conditions that do not allow a period of rest (dwell time) between load applications. Hence, when laboratory test conditions incorporate a pulse load, resulting in an

applied load plus a rest period for each cycle, a unique fatigue will not be obtained for all temperature and load rate conditions. This concept is also verified by studies reported by Van Dijk (see Figure 40) which show the variable (increasing) effect of rest periods upon fatigue. Extending this concept, it can also be surmised that the effect of rest periods will be different for differing temperatures.

As a result of this study, it is the author's opinion that for conditions similar to those found for AC mixtures being repeatedly stressed by vehicular loads in the field, that the concept of a "unique strain criterion" is not applicable. A more accurate prediction of the fatigue response may be obtained from laboratory testing using a pulse type of load condition. However, much research is still needed on the effects of rest time upon fatigue behavior, especially at variable (primarily high) temperature conditions.

NONPARALLEL VERSUS PARALLEL MULTITEMPERATURE FATIGUE CURVES

In the last section, the validity of the unique fatigue relationship was found to be probably due to the use of continuously applied loads in fatigue testing. For a more accurate simulation of traffic in the field, the use of a pulse-type load was found to be more suitable, but also resulted in multistiffness (temperature) fatigue curves. For those researchers reporting temperature- or stiffness-dependent fatigue relationships, a difference occurs as to whether these relationships appear as a series of parallel (or close to) curves or whether they are nonparallel and intersect at some mutual point.

The researchers or organizations having multitemperature relationships have been noted to be Monismith, Kirk (impulse), TAI (laboratory tests and Kingham criteria), Shell Oil Company (original fatigue curves), and Coffman. Like the previous section, dealing with the validity of the strain criteria concept, an examination of the major factors in the testing conditions was done by the author for each of the researchers/organizations noted to report multistiffness (temperature) curves.

Table 10 is a summary comparison illustrating strong evidence

Table 10

Comparison of Types of Multistiffness Fatigue Curve Results

Researcher/Organization	Figure No. Reference	Type of Fatigue Curves	Nonlinearity Accounted for in Determining AC Modulus
Monismith (CA)	19	Nonparallel	Yes--direct deflection measurement of specimen
TAI (lab results)	31	Nonparallel	Yes--direct deflection measurement of specimen
Coffman (Ohio State)	51	Nonparellel	Yes--direct strain gages on specimens
Kirk (impulse)	30	Parallel	No--Shell nomograph used
TAI (Kingham)	34,35	Parallel	No--direct lab test with dynamic modulus test
Shell Oil (original)	37	Parallel	No--Shell nomograph used

supporting the hypothesis explaining the reasons for obtaining either parallel type or nonparallel multistiffness fatigue curves. It is believed to be strongly related to whether or not the nonlinear stress behavior of asphaltic concrete is taken into account during the fatigue tests in the calculation or determination of the tensile strain in the mix. Many researchers have shown beyond a doubt that asphalt concrete, especially at high temperatures, is very nonlinear.

For the three researchers reporting somewhat strong nonparallel fatigue relationships (Monismith, TAI Laboratory, and Coffman), it can be observed that the strains were either directly measured by strain gages or calculated from flexural stiffnesses E obtained from direct specimen deflection measurements. In each of these cases it is noted also that the nonlinear (stress dependency of the modulus) behavior is directly accounted for in the testing phase.

For the three agencies that reported parallel-type fatigue curves, the common link is that each of them calculated tensile strains with a modulus that is, in itself, linear and does <u>not</u> account for any stress dependency. The indirect nomographic solution for stiffness using the Shell nomograph considers only the effect of bitumen penetration index, ring and ball temperature, and frequency of load. It does <u>not</u> account for the known nonlinear behavior, especially at elevated temperatures. Finally, even though the dynamic modulus is a direct laboratory test, the test conditions almost generally result in such low stress levels at high temperatures as to render the test linear, rather than nonlinear.

The technical explanation as to how nonlinearity affects the fatigue results has previously been hypothesized and demonstrated by Witczak[45]. In essence, because fatigue tests in the laboratory generally are conducted under large stress levels so that fracture may occur in a relatively short test time, nonlinear stiffnesses are introduced in the test (primarily for flexural-type conditions at high temperatures). Thus, if the nonlinear stiffness is used to calculate the strains from the applied tensile stress the fatigue curve established will be much steeper than one obtained using a linear modulus.

Based upon the observations noted in this and the previous

section, it would appear that asphaltic concrete fatigue curves are probably best represented by a family of multimodulus (stiffness) curves that are parallel or nearly so. In addition, it can also be stated that powers (slopes) of the E curves obtained with linear moduli will be greater (in magnitude) than those obtained from nonlinear characterization. However, as noted previously, more fundamental research is required to verify and quantify the results of these concepts of rest time and methods of determining strain and temperature upon the fatigue results.

LABORATORY-FIELD ADJUSTMENT FACTORS

It has already been stated that, due to several reasons, the use of direct laboratory results within a fatigue subsystem will offer the most conservative estimate of failure. Among these are differences due to vertical crack propagation time, rest time effect between pulses, possible effect (beneficial) of healing, use of simple loading in lieu of sequential random loading, failure of laboratory tests to properly account for high temperature effects upon crack propagation rates and finally, the time or repetition effect for initial surface cracking to cause "functional" failure of the pavement system.

In addition to these complexities, it should also be recalled that there are several current fatigue design criteria available. This, coupled with the unknown effects between various materials, test procedures, and different types of damage models used, the unknown effect of subgrade type upon crack propagation, and the thickness effect all leave open the question as to the validity of a "unique" laboratory-to-field adjustment factor applicable to all criteria, that can be used to accurately predict "failure" in a design subsystem.

Fortunately, there exists a fairly good source of verification research that can be summarized to establish general ranges of factors that can afford valuable insight into the design problem. In general, almost all of these verification studies have dealt with "phenomenological" fatigue studies rather than "mechanistic." Most of these efforts

have also been made on thick (e.g., full-depth) asphalt concrete pavement sections, thus tending to decrease the confidence level of the noted values for granular base flexible pavements.

It should also be recalled that several agencies have postulated various adjustment factors to be applied to laboratory fatigue tests and/or typical provisional types of criteria. Among these are values of 5 (Brown), 20 (Brown and Pell), 100 (Brown), and 10 (Shell Oil). Thus, these postulated values have ranged from 5 to 100. Table 11 is a condensed summary of adjustment factors (in reality, safety factors) that have been obtained from various researchers while doing verification-type studies on asphalt fatigue studies. Also shown in the table is a general description of the test conditions (e.g., laboratory or field study, direct laboratory fatigue results, or typical criterion etc.), as well as how "failure" was reported in the referenced research work.

From the table it can be observed that most of the determined ratios conducted under more controlled laboratory-type conditions generally yield conservative values. This is in contrast to several field studies where ratios less than 1 have been recorded. It is the author's opinion that the major reason for this is the extreme difficulty in accurately defining a representative subgrade/base modulus to be used in the analysis. In addition, most of the computed values can be seen to be much less than the postulated ratios of 5 to 100.

Based upon a review of the table, the author suggests that an adjustment factor of 2.5 be used for "initial surface crack" conditions, and a ratio of 5.0 be used for "functional-type failure" when using phenomenological, multistiffness fatigue curves for thick asphalt sections. It is probably true that these ratios should be approximately the same for granular base sections; however, because of the added difficulty in characterizing other base layer moduli, the suggested values should be about 1.5 (initial cracking) and 3.0 (functional). Use of these ratios would appear to be slightly conservative.

When using a unique "strain criterion" as proposed by Pell and others, it would appear that larger ratios would be applicable because of the already mentioned conservatism (design) inherent in the use of

Table 11

Summary of Fatigue Verification Studies

Test Conditions*	References	Basis for "Failure"	Safety Factor (Observed to Predicted Repetition Ratio)	Remarks
L-D-M-1	Majidzadeh[87]	Surface Crack--Initial	FS = 1.0	Average of several tests
L-D-P-2	Yuce-Monismith[28]	Bottom Crack--Initial	FS = 1.7 (0.5-3.8)	
L-D-P-2	Coffman[79]	Surface Crack--Initial	FS = 2.7 (1.7-3.5)	
L-D-M-2	Majidzadeh[87]	Surface Crack--Initial	FS = 0.95-1.5	Based upon Majidzadeh's N_f
L-D-M-2	Majidzadeh[87]	Surface Crack--Initial	FS = 4.7	Based upon Coffman's N_f
L-D-P-3	Bazin-Saunier[64]	Surface Cracking	FS = 2-4	Circular test track results
L-D-P-3	Van Dijk[74]	(See remarks)	FS = 3	Ratio from bottom to surface cracking
		(See remarks)	FS = 3-6	Ratio from surface to functional failure
		(See remarks)	FS = 10-20	Ratio from bottom cracking to functional failure
F-D-P-3	Kingham[44]	Surface Crack--Initial	FS = 0.9 (0.8-1.1)	Lab constant stress-sand asphalt
			FS = 0.85 (0.8-0.9)	Lab constant strain-sand asphalt
			FS = 0.71 (0.31-0.92)	Lab constant stress-asphalt concrete
			FS = 0.72 (0.3-0.94)	Lab constant strain-asphalt concrete
			FS = 0.26 (0.08-0.32)	Lab constant stress-crushed stone
			FS = 0.26 (0.08-0.30)	Lab constant strain-crushed stone
F-D-P-4	Monismith[20]	Bottom Crack--Initial	FS = 4.7	90 percent confidence level on fatigue curve
			FS = 2.1	50 percent confidence level on fatigue curve
F-D-P-4	Witczak[45]	Surface Crack--Initial	FS = 5.1	Dynamic modulus used for fatigue
			FS = 0.9	Average flexural modulus used for fatigue
			FS = 1.1	Stress-dependent modulus used for fatigue
		Functional Failure	FS = 7.5	Dynamic modulus used for fatigue
			FS = 1.4	Average flexural modulus used for fatigue
			FS = 1.6	Stress-dependent modulus used for fatigue
F-T-P-4	Kingham criteria[45]	Surface Crack--Initial	FS = 1.28	Dynamic modulus used for fatigue
			FS = 0.73	Average flexural modulus used for fatigue
			FS = 0.53	Stress-dependent modulus used for fatigue
		Functional Failure	FS = 1.89	Dynamic modulus used for fatigue
			FS = 1.09	Average flexural modulus used for fatigue
			FS = 0.79	Stress-dependent modulus used for fatigue
F-T-P-4	Kentucky criteria[45]	Surface Crack--Initial	FS = 0.2	Dynamic modulus used for fatigue
			FS = 0.7	Average flexural modulus used for fatigue
			FS = 1.1	Stress-dependent modulus used for fatigue
		Functional Failure	FS = 0.4	Dynamic modulus used for fatigue
			FS = 1.0	Average flexural modulus used for fatigue
			FS = 1.7	Stress-dependent modulus used for fatigue
F-T-P-4	Monismith criteria[45]	Surface Crack--Initial	FS = 0.7	Dynamic modulus used for fatigue
			FS = 1.3	Average flexural modulus used for fatigue
			FS = 1.5	Stress-dependent modulus used for fatigue
		Functional Failure	FS = 1.0	Dynamic modulus used for fatigue
			FS = 1.9	Average flexural modulus used for fatigue
			FS = 2.2	Stress-dependent modulus used for fatigue
F-T-P-3	Kingham[44]	Surface Crack--Initial	FS = 20.4 (3.3-40.0)	Field AASHTO modified-sand asphalt
			FS = 0.9 (0.8-0.95)	WSU modulus-sand asphalt
			FS = 8.7 (2-14)	Field AASHTO modified-asphalt concrete
			FS = 1.8 (1-3.2)	WSU modulus-asphalt concrete
			FS = 98.0 (30-122)	Field AASHTO modified-crushed stone
			FS = 11.6 (3.8-144)	WSU modulus-crushed stone

* Test condition legend: Location of experiment, L = lab, F = field; type of fatigue criteria, D = direct test; T = typical criteria; fatigue method, P = phenomenological, M = mechanistic; test condition: beam on elastic foundation (1), prototype pavement slab (2), test track sections (3), actual pavement performance (4).

such a laboratory criterion over multistiffness fatigue curves.

In general summary, it can be seen that, by and large, conservative estimates of fatigue life can be obtained by using phenomenological laboratory tests in a fatigue design subsystem. It is suggested that, wherever possible, the laboratory fatigue tests be determined on the actual job mix to be used. Suggested ratios to account for both laboratory to field initial cracking differences as well as initial cracking to some "functional" failure condition have been stated. It is cautioned that these factors are probably dependent upon many other variables, particularly with the type of fatigue conditions used in the test and type of subgrade. As such they should be considered as provisional types of guides. Obviously, much more verification work is needed to truly assess what the proper ratio is for any given fatigue model and field conditions. Thus, much more research is necessary to define the true relationship between distress and performance due to cracking.

EFFECT OF MIX VARIABLES

Several researchers have studied in detail the effect of mix variables upon laboratory fatigue behavior. Among these are Pell and Brown, Monismith, Verstraeten, Kirk, and Bazin and Saunier. For conditions of constant stress testing, it appears that the two major factors affecting the strain-repetition curve are related to the volumetric percentage of bitumen and voids in the mix and some characteristic of the bitumen used in the mix.

Relative to the first factor, Pell has adopted the volume percentage of binder V_B directly, Monismith uses the void volume percentage V_v and mix stiffness, Bazin and Saunier use both V_v and V_B while Verstraeten and Kirk have selected the volumetric ratio of bitumen to voids in the mineral aggregate as the parameter of interest. To account for bitumen properties, the ring and ball temperature has been suggested by Pell (note that this also affects the mix stiffness used by Monismith), while the asphaltene content has been suggested by both Verstraeten and Kirk. It should also be recalled that Verstraeten has correlated asphaltene content to ring and ball temperature as well as bitumen viscosity.

While there is no universal agreement as to the precise parameters to be used, the results do show that there are only several major mix variables that will affect the strain-life relationship. Thus, such factors as aggregate size, gradation, and type of aggregate appear to exert a very small influence on fatigue behavior.

An important consideration to be noted is that there is universal agreement (among researchers conducting phenomenological laboratory studies) that in order to increase the fatigue life of asphalt under constant stress conditions, the mix stiffness should be as large as possible. Although this result has been observed repeatedly, it should be realized that it does conflict with other fatigue criteria such as Kingham's field-developed model, Shell provisional curves, and, to some limited extent, mechanistic studies. The state of the art is such that it must be concluded that there is no field evidence to firmly establish what the true effect of changing mix stiffness will be relative to field performance. Because of the many factors already noted that affect the laboratory-to-field fatigue performance of asphaltic mixtures, it is the author's opinion that the selection of a stiffer mix may not increase the field life, even for thicker asphalt sections. Accordingly, it is felt that more research should be conducted to verify the actual effect of mix stiffness upon field fatigue performance.

ASPHALT EMULSION FATIGUE

Regarding the general state of the art relative to fatigue of asphalt emulsions, almost everything that has been stated about both the General Results and Asphalt Concrete Fatigue sections of this chapter are likewise applicable to emulsions. However, regarding a design subsystem for emulsion fatigue, an added complication arises because of the time dependency of the mix, due to curing of the volatiles. It appears that the use of small percentages of cement (cement-modified emulsion) allows the material to behave in a pattern much more similar to an asphalt concrete. It would thus appear that for such materials the confidence level for fatigue analysis would be similar to that of apshalt concrete.

The fatigue curves developed by Chevron Asphalt and shown in Figures 57 and 58 have already been adjusted by a factor of 3.0. Based upon the section of this report entitled Laboratory-Field Adjustment Factors, it would appear that such a value is very reasonable to use at this time for design. However, the recommended design procedure by Chevron still needs much verification of actual field performance results, particularly of the postulated mechanism for accounting for the increase in stiffness with time due to curing of straight emulsion mixes. As a result the overall state of the art for emulsions would be considered as being poorer than that for asphalt concrete fatigue.

LIME-TREATED MATERIAL FATIGUE

There is little information on the fatigue of lime-treated materials. The only information available appears to be concerned with several laboratory fatigue studies conducted in a phenomenological mode. The development of a fatigue subsystem is lacking in the literature although such a system could be easily developed with current technology.

Like the asphalt emulsions, lime-treated materials show a gain in strength with time due to curing. Little is known of the ability to adequately predict the change in fatigue behavior in situ with time, as well as in the moduli. An added complicating factor also arises due to the high probability of having this stabilized layer placed in improved subgrade or subbase type layers. For these conditions, there is an added problem of ascertaining the effect upon surface cracking or roughness. This appears to be an extremely important determination for which there is little, if any, information. It would appear that much more fundamental research, as well as verification-type studies, is needed to fully develop a fatigue subsystem for this material.

FLY ASH-TREATED MATERIAL FATIGUE

The use of fly ash with lime and lime-cement additives to form a stiff stabilized layer is relatively new (although the concept has been known for many years). These material types take on more of the performance characteristics of PCC pavements and, as such, show a great

deal of similarity to PCC fatigue behavior. However, as in lime-treated soils, the effect of increasing characteristics (moduli and fatigue behavior) with time due to curing is an important feature that must be recognized in any fatigue-performance subsystem.

Because of its slablike properties, little information regarding the applicability of an elastic layered theoretical model is known. In most studies, slab theory has been used to predict performance of this material. However, as for almost all other materials reported in this publication, verification studies relative to the applicability of a fatigue subsystem to predict cracking (load associated) are needed.

CEMENT-TREATED MATERIAL FATIGUE

In the past several years there has been a great deal of research effort expended to develop a more fundamental design approach to cement-treated materials. As for lime-treated, fly-ash treated, and PCC materials, fatigue behavior has been basically developed from stress to strength ratios. Monismith has recently shown that such behavior in fatigue can also be interpreted in the strain-repetition mode, as is usually done for asphaltic materials.

One of the major problems associated with the development of a fatigue subsystem for cement-treated materials appears to be related to the selection of an appropriate modulus of elasticity to be used in the fatigue analysis. This problem is mainly brought about by the known cracking that will occur due to shrinkage in these layers before loads are even applied. Thus, the "secondary" cracking brought about by fatigue loading is the more important to the performance-time relationship. However, another significant problem within a fatigue subsystem occurs with the analysis of stresses and strains in a cracked (due to shrinkage) layered system. Although Westergaard's slab theory has been used in much analysis, the successful application of elastic layered theory and finite-element approaches has been demonstrated by Monismith.

Thus, the use of laboratory moduli along with finite-element or elastic layered theory has been shown to yield good agreement between measured strains and stresses. With this information, it would appear

that the major source of necessary research is that related to verification studies of predicted cracking within the fatigue subsystem.

PORTLAND CEMENT CONCRETE FATIGUE

The state of the art for PCC fatigue probably is the best for all the materials noted. While several laboratory results still need investigation, the biggest research efforts are needed in the general categories of accounting for the stochastic nature of fatigue results, as well as determining the distress to performance model.

GENERAL SUMMARY

Table 12 is an overall summary of the estimated state of the art of fatigue for the eight materials presented herein. The summary is shown for eleven different factors related to an overall fatigue subsystem that uses a performance model (i.e., functional rather than structural failure). The ratings shown are based on the author's subjective qualification and as such may differ from those of other individuals.

It is the author's opinion that the fatigue subsystem state of the art is best for PCC and asphalt concrete materials. The poorest state of the art appears to be for lime-treated materials and asphalt emulsions. The average rating shown indicates that the state of the art for the LTS materials is "fair" while the rating for PCC and AC materials is "good." Much of the reason for the somewhat high rating of the cement-modified emulsion (CMAE) given by the author is a direct result of the similarity of its behavior to asphalt concrete mixes.

The table also indicates the average state of the art relative to factors within the fatigue subsystem. In general, it is the author's opinion that the state of the art relative to the selection of a theoretical model, material characterization techniques, and ability to conduct laboratory fatigue tests is generally good. However, the poorest factors, exclusive of material type, relate to the consideration of the stochastic nature of fatigue results, the ability to incorporate a distress to performance model within the subsystem, and the need for more extensive verification studies of the fatigue methodologies.

Table 12

State-of-the-Art Summary, Fatigue Subsystem

Factor	Material Type AC	AE	CMAE	LTS	LFA	LCFA	CTM	PCC	Average Rating	Rank by Factor
a. Theoretical stress/strain prediction	5*	4	5	5	4	4	4	5	4.5	(1)
b. Ability to evaluate material modulus	5	3	4	4	4	4	3	4	4.0	(2)
c. Lab fatigue testing	4	3	4	3	3	3	4	5	3.6	(3-4)
d. Effect of mix variables (lab conditions)	5	4	3	2	3	3	3	5	3.5	(5-6)
e. Effect of mix variables (field conditions)	3	2	3	2	3	3	3	4	2.9	(8-9)
f. Ability to consider cure or age effects in fatigue analysis	5	2	4	3	3	3	4	5	3.6	(3-4)
g. Development of fatigue subsystem	5	3	3	2	3	3	4	5	3.5	(5-6)
h. Reliability to estimate initial structural crack	4	2	3	1	3	4	3	5	3.3	7
i. Verification of fatigue subsystem	4	2	3	1	3	2	3	5	2.9	(8-9)
j. Distress to performance relationship	2	2	2	1	2	2	2	3	2.0	(10)
k. Consideration of stochastic properties	2	1	1	1	1	1	1	2	1.3	(11)
Average	4.0	2.5	3.2	2.3	2.9	2.9	3.1	4.5		
Rank by material	(2)	(7)	(3)	(8)	(5-6)	(5-6)	4	(1)		

* Legend: Poor - 0
Fair - 2
Good - 4
Excellent - 6

RECOMMENDATIONS FOR FUTURE RESEARCH

GENERAL FATIGUE RESEARCH STUDIES

Item 1. It is the opinion of the author that a fatigue subsystem developed for all pavements should be based on a universally acceptable form of functional failure rather than on structural distress (cracking). As a result, much research must be done. More specifically, these items are related to the following.

a. The level of performance that constitutes the terminal or functional failure condition for any pavement type must be defined.

b. Because all fatigue methodologies are based upon distress prediction (i.e. some form of crack initiation or areal percentage of cracking), it is equally as important to conduct research to establish the distress to performance relationship for any pavement type.

c. Inherent in accomplishing b is the ability to define the pertinent variables as well as to quantify their influence upon the subsequent rate of crack propagation on serviceability loss rate. Such research will then truly define a fundamental approach to pavement fatigue.

Item 2. The results of this study illustrated that the fatigue response of either laboratory material specimens or field pavement section performance is highly variable. This random or stochastic effect is so significant that it appears impossible to truly have a fundamental fatigue design subsystem unless this variable is incorporated into the analysis. Accordingly, a research effort is needed to develop a probabilistic fatigue design system.

Item 3. Even though there remain several types of needed laboratory studies that concern fatigue behavior, it is a general overall observation of the author that much more research should be focused upon field verification of existing fatigue methods. It appears that, regardless of material type, there exists, at least, a basic foundation of knowledge for an analytical fatigue subsystem. Such models may and do depend upon the material type, but at least they currently exist. It is the author's opinion that full confidence in such methods by the

profession will never be obtained until significant future research (relating directly to the verification of such models by field performance studies) is made. It is also felt that much more research will be required for bituminous and other stabilized materials, in contrast to that required for PCC pavements.

Item 4. Based upon the current state of the art, it appears that phenomenological fatigue testing (independent of material type) should be continued to develop and verify fatigue design subsystems. However, because of distinct advantages relative to the mode of loading effects, crack propagation rates, etc., future research should also be continued to develop a mechanistic approach based upon fracture mechanics principles. It is felt that such a method will afford many more advantages for determining asphaltic material fatigue, than for the other stabilized material types. This is primarily due to the uncertainties that presently exist regarding the effect of temperature upon fatigue life. Studies have indicated that the critical crack depths are very small (≈0.10 in.) for PCC and other pozzolanic materials. This implies that fracture is almost instantaneous (which is obviously true for these materials). Thus, little added knowledge relative to the rate of crack propagation could be obtained for these materials. There is no comparable compilation of such information for bituminous materials. This feature, plus the many years of successful performance based studies using the phenomenological approach, appears to warrant a continuation of the current approach for "rigid" materials.

SPECIFIC MATERIAL FATIGUE RESEARCH STUDIES

ASPHALT CONCRETE

Item 5. Based upon the extensive studies presented by several researchers, it is the opinion of the author that there is absolutely no need to conduct any further extensive laboratory fatigue tests to ascertain the effect of mix properties upon fatigue life. However, it has been shown that currently there does exist in the literature a difference of opinion as to how extreme mix stiffness (either high or low)

affects fatigue performance. In general, all phenomenological laboratory testing results have indicated that increasing mix stiffness increases fracture life. This is in contrast to the existing field derived criterion developed for functional failure conditions (terminal serviceability level) by Kingham. Because of this difference, it is highly recommended that future research studies be conducted with field performance tests (not laboratory or theoretical analysis) to conclusively determine if cold weather-high stiffness mixes are more or less damaging than warm weather-low stiffness mixes.

Item 6. It is the opinion of the author that one of the most salient laboratory studies needing research in asphalt fatigue relates to defining the effect of various pulse load (rest periods) conditions at various temperature levels upon phenomenological fatigue life. It is strongly felt that such a study will shed light into the contrasting results of researchers regarding the validity of the "strain criterion" postulated by Pell and others. It is postulated, based upon a summary review of various fatigue research studies, that if pulse loads are used in the fatigue test, multistiffness curves will result (rather than a unique fatigue curve which appears to be directly related to the use of continuously applied load conditions).

Item 7. Another major laboratory research study should be initiated to confirm the importance of accounting for the nonlinear behavior of asphaltic mixes during fatigue testing. A set of parallel or nonparallel type of fatigue curves should be developed. Evidence noted in this report strongly suggests that when linear stiffnesses are used to compute tensile strains in fatigue tests, a set of parallel-type fatigue curves results. However, when the nonlinearity is accounted for to assess the strain, a set of strongly nonparallel types of fatigue curves will result. This trend has been repeatedly verified by various researchers when pulse loads have been used in the fatigue testing (see Item 6 above).

Item 8. At present there exist several different postulated fatigue criteria that have been developed for "typical" mixes. Although

these systems have been shown to all give reasonable estimates of fatigue life, it is virtually impossible to state which criterion is the most accurate. If it is desired to continue using such an approach (i.e., use of a "typical" or "provisional" type of fatigue criterion rather than direct laboratory testing) then future research will be needed to obtain many verification results with each criterion. It is the opinion of the author that only in such a way will knowledge be obtained to provide information as to the true accuracy of each procedure. If such research is conducted it is recommended that use of a cumulative damage approach with multistiffness fatigue curves be employed rather than the nebulous selection of an "effective" or "critical" modulus condition. It is also recommended that the basis for such a study employ field performance studies rather than laboratory tests.

Item 9. In this report, the great differences in fatigue behavior between various asphalt mixes have been repeatedly demonstrated. For this reason alone, it is the strong recommendation of the author that any fatigue subsystem developed in the **future** should rely strongly upon the use of laboratory fatigue tests conducted on the actual job mix rather than on the use of "typical" or "provisional" fatigue criteria. Furthermore, these laboratory tests should conform to the results obtained from the added knowledge of research Items 6 and 7. Although there exist several procedures that can predict, reasonably accurately, the laboratory fatigue behavior, all of these procedures have been developed for conditions of the "unique strain criterion" hypothesis. Accordingly, the author feels that the current use of such procedures would be misleading (although conservative). Therefore, it is recommended that research be conducted to firmly establish the appropriate adjustment factors to account for initial surface cracking conditions, various percent areal cracking levels (e.g., 10, 20, 30, and 40 percent), as well as for functional failure conditions that can be applied to direct laboratory testing. Again, it is strongly recommended that cumulative damage techniques be used and that the basis for such a study be field performance based.

ASPHALT EMULSION

Item 10. Many of the research suggestions for asphalt concrete are also applicable for asphalt emulsions. However, it has been stated, that the added variable of curing strongly enters into the fatigue of emulsified asphalt mixes. Accordingly, it is the opinion of the author that one of the most critical research needs is the verification of the methodology suggested in this report by Chevron Asphalt Company regarding the development of the mix stiffness with time and temperature for emulsion. In particular, the research objective should be to verify the correlation of the suggested laboratory approach to moduli determined from field specimens.

Item 11. Typical fatigue curves for various mix stiffnesses have been presented in this report for "typical" emulsion mixes. It is the opinion of the author that more verification research should be conducted to conclusively establish that such curves properly account for the effect of variable curing conditions upon fatigue behavior. In other words, are the same set of fatigue curves applicable for all combinations of cure percentage and temperature that result in the same mix stiffness?

LIME-TREATED MATERIALS

Item 12. The state of the art for development of a fatigue subsystem for lime-treated soils is probably one of the poorest among all other stabilized material types. It is felt, however, that this rating is, in all probability, directly related to the infrequent use of this stabilized material in pavement layers that would normally be subjected to conditions conducive to fatigue loading and resultant fracture. As a result, the need for research would appear to be related to the anticipated future use of this material in the upper (base) layers subjected to high stresses from loads. However, presuming this be the case, it appears that much fundamental laboratory fatigue work is needed to assess the influence of mix variables upon fatigue life. In conjunction with such a study, it would also appear that research is needed to

verify the change, if any, in fatigue behavior with variable degrees of curing.

Item 13. In order to properly develop a fundamental fatigue system for this material type, it appears that research is needed to accurately assess the change in structural pavement response brought about by complete fatigue (fracture) of the stabilized lime layer. For this, and other treated materials, that will perhaps be placed in lower pavement layers, the factors influencing the rate of vertical crack propagation to the surface need to be carefully evaluated.

FLY ASH-TREATED MATERIALS

Item 14. In many instances, the behavior and performance of stabilized layers using fly ash and other additives are quite similar to cement-treated and PCC materials. However, because the mix composition is important, it appears that some fundamental research is necessary to assess the influence of these parameters upon the fatigue behavior of the material.

Item 15. In conjunction with Item 14, it is suggested that research be conducted (or possibly an extensive literature review) to accurately define the effect of cure time upon the failure repetition-stress to strength ratio fatigue plot. While conducting such a study, it is imperative that stress to strength ratios be established from strength measurements conducted on specimens having the same cure time as when being fatigued.

Item 16. Historically, theoretical studies with this material have used slab theory to compute stresses and strains. Although these results have been satisfactory to date, it is suggested that the use of elastic layered theory be investigated to determine the validity of this methodology for stress and strain prediction. There is a reasonable and satisfactory comparison of predicted and observed parameters with layered theory for other materials (e.g., asphalt concrete, emulsions, lime-treated soils, and even cement-treated bases). The satisfactory use of this theory with this stabilized material type would greatly

enhance the ability to use one general method of fatigue analysis that would be directly applicable for all stabilized materials, exclusive of PCC pavements.

CEMENT-TREATED MATERIALS

Item 17. One of the most critical items that has to be researched in regard to fatigue of cement-treated materials is the evaluation of the proper material modulus to be used in the fracture subsystem. Various researchers have suggested conflicting ways or procedures that can properly consider this factor. However, they have ranged from using a moduli comparable to that of an unbound granular material to the use of the direct laboratory measured dynamic modulus from a lab specimen. The selection of the modulus value, with the aforementioned range in magnitude, will probably constitute the single most important variable to accurately predict fracture life of this material.

Item 18. In addition to the above-noted item, continued research should be focused upon the development of an analytical procedure (i.e., type of theory, effect of shrinkage crack, jointing, etc.) that most satisfactorily allows the prediction of fracture upon repeating loading. Recent research has shown that elastic layered theory may be used for this purpose. However, it is felt that more verification of such an approach is warranted.

Item 19. Historically, much of the initial laboratory fatigue testing of this material has employed the concept of curvature, rather than strain or stress parameters, to express fatigue life. Recent studies have suggested that use of either of these parameters (stress, strain, or curvature) is somewhat comparable. As a result, further research appears warranted to determine the validity of using either strain or stress directly within the fatigue characterization law.

Item 20. It is recommended that further research be developed to assess the fatigue behavior of this material as a function of cure time. Such a study should compare and evaluate fatigue results based upon stress, strain, and stress to strength ratio. Like other such

suggested research studies in this section, stress/strength ratios should be established using the actual strength at the specific cure time as when the specimen is fatigued.

Item 21. Although not directly related to fatigue per se, it is felt that continued research should be done to develop either some additive or construction procedure that can greatly minimize or eliminate shrinkage cracking in cement-treated materials. The development of such a material or technique would greatly reduce the predictive problems associated with the theoretical stress-strain analysis within any proposed fatigued subsystem.

PCC

Item 22. It is the author's opinion that the state of the art for PCC fatigue is more advanced than all other material types considered in this report. As such, the major research problems associated with PCC fatigue are those presented in Items 1-4.

Item 23. Several minor laboratory studies should be investigated to confirm or invalidate the hypothesis of Kesler regarding the existence of a unique stress to strength ratio versus repetition relationship for PCC material.

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